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STABILIZATION OF THE MIDDLE RIO GRANDE IN NEW MEXICO

By Robert C. Woodson[1]

SYNOPSIS

The Middle Rio Grande in New Mexico comprises a reach of the river approximately 183 miles long. The Rio Grande floodway confines the river throughout most of this reach.

In the Cochiti-Rio Puerco unit, the Rio Grande occupies a wide shallow channel between the levees of the floodway. The channel has no banks and the average level is generally at or above the level of the areas behind the levees.

The levees, constructed as a part of the floodway, are subject to attacks by the river. Early attempts to protect the levees by means of pile jetties demonstrated the need for a more effective system of levee protection. Deterioration of the floodway due to aggradation and the high losses of water which were occurring indicated the need for channel stabilization.

Channel stabilization will be provided throughout most of the floodway in the interest of flood control, major drainage, and water salvage. The Cochiti to Rio Puerco unit of the floodway is under construction (in 1961). This work includes levee rehabilitation and stabilization of about 105 miles of channel. The Kellner Jetty System is being utilized in the channel stabilization works.

NATURAL CHARACTERISTICS OF THE RIVER

The Middle Rio Grande in New Mexico begins near the Cochiti Pueblo about 35 miles above Albuquerque, New Mexico and extends downstream to the

Note.—Discussion open until April 1, 1962. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 87, No. WW 4, November, 1961.

[1] Chf., Engrg. Div., U. S. Army Engr. Dist., Albuquerque, N. Mex.

upper limits of Elephant Butte Reservoir, a distance of about 183 river miles. The river is generally confined by the Rio Grande Floodway which includes levees, channel rectification, and stabilization, throughout most of this distance. The floodway was constructed by the Middle Rio Grande Conservancy District in the early 1930's to provide flood protection to the adjacent irrigated areas and urban developments. It is now being rehabilitated by the Corps of Engineers and the United States Bureau of Reclamation Dept. of Interior (USBR). A part of the plan of rehabilitation provides for a stabilized channel within the floodway.

The floodway has been sub-divided into the Albuquerque unit, the Cochiti-Rio Puerco unit, and the Rio Puerco-Elephant Butte unit. Selection of these units or reaches was based primarily on the drainage pattern and extent of development within the Middle Rio Grande Valley. The Cochiti-Rio Puerco unit is the longest. It extends downstream from Cochiti to the mouth of Rio Puerco, a distance of about 105 river miles and includes the 20 mile Albuquerque unit. The Albuquerque unit was treated separately because of the large development adjacent to the river which required special consideration for flood control. The Rio Puerco-Elephant Butte Unit extends downstream from the mouth of the Rio Puerco to the upstream end of Elephant Butte Reservoir. The Rio Puerco was selected as the dividing point between the two units as it is the largest tributary entering the river below Albuquerque and it exerts a significant effect on the regime of the river below the confluence. Because the Cochiti-Rio Puerco unit including the Albuquerque unit is the longest and most important, description of the natural characteristics of the river and subsequent description and comment will be confined to this unit or reach. Fig. 1 shows the location of the floodway within the Middle Valley.

Flows in the Middle Rio Grande are of two general types. One type occurs in the spring, from April through June, as a result of snow melt that is often augmented by general precipitation. The other is the summer flow or flash flow which occurs from May through October as a result of rainfall. The spring flows are characterized by a gradual rise to a comparatively moderate rate of discharge that is usually maintained for about two months with peak flows of shorter duration. Volume of runoff is large. Summer flows rise sharply to a peak and recede rapidly. The volume of runoff is generally small; however, the peak flows are comparatively large and travel downstream as a wave that flattens as floodway storage becomes effective. Flows above 5,000 cfs are considered flood flows. Table 1 indicates the flows within the Cochiti-Rio Puerco reach based on available records at Bernalillo, but without considering regulation to be provided by dams. Records of stream flow on the Rio Grande began in 1889. Historically, it appears that much greater flows have been experienced than indicated by Table 1. Records left by a Catholic priest at Tomé, New Mexico, located about 28 river miles downstream from Albuquerque, regarding the May-June 1828 flood were the basis of an estimate by the International Boundary and Water Commission that the peak flow may have been as high as 100,000 cfs. Early newspaper accounts describe extensive damage in the Middle Valley from the spring floods of 1865, 1874, and 1884. The peak of the 1874 flood was estimated from records of high water marks at discharges ranging from 45,000 cfs to 125,000 cfs.

The slope of the river is approximately 5 ft per mile from Cochiti to a point just below Albuquerque and about 4 ft per mile from this point to the Rio Puerco. As previously mentioned, the river is generally confined by levees

throughout the reach. With the exception of the upper 22 miles above Angostura Diversion Dam, the floodway averages about 1,500 ft in width. Above Angostura the levees are discontinuous and the width of the floodway varies erratically from about 500 ft to 2,500 ft. In this area, property values are low and a continuous levee system was not justified. The river flows in a wide shallow channel of variable width except during floods when practically the entire floodway width is covered with flood waters. Below Angostura Diversion Dam, the channel averages about 800 ft in width. The average height of the levees is about 8 ft. Between the channel and the levees there are growths of salt cedar, cottonwood, and willows of varying density, commonly called bosque. For all practical purposes, the channel has no banks and is generally at or above the level of the valley behind the levees.

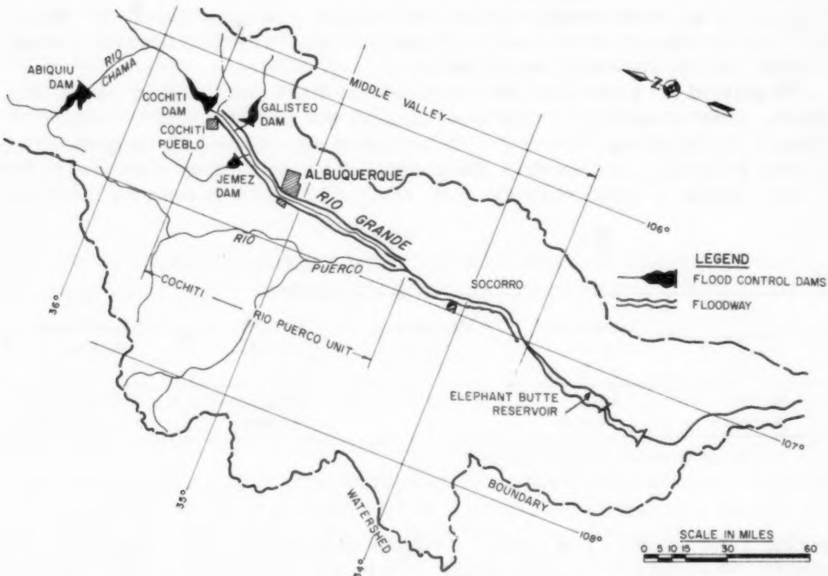

FIG. 1.—MIDDLE RIO GRANDE VALLEY IN NEW MEXICO

The stages and discharges in the Rio Grande within the Middle Valley are not closely related. This is due to the easily eroded streambed material which, because of variable channel scour, produces different stages for the same discharge. Considering the maximum floods of record, the stages have varied between about 4 ft to 6 ft above the average channel elevation prior to the flood.

The suspended sediment flow of the river is large. At Cochiti it averages about 0.194% by weight and at Bernado, located a short distance above the mouth of the Rio Puerco, it averages 0.454%. The percentage of sand in the suspended sediment increases rapidly with increase in flow. For example, the sand in samples at Cochiti average about 3% for flows less than 500 cfs and about 34% from flows between 2,000 cfs and 3,000 cfs. There is more sediment

entering the floodway than is being removed, and as a result the floodway is aggrading at an average rate of about 3 ft every 50 yr.

The material of the channel bed is principally sand with the finer sizes predominating. Generally, less than 10% of the material is silt and clay. There is some fine gravel present but the amounts are not significant.

The configuration of the river may be described as one of low sinuosity with some straight reaches. The ratio of length of channel to length of valley is approximately 1.1 to 1. The channel is braided which is probably due to the combination of relatively steep slope and overload of sediment. The basic levee system, which was completed in 1936, was laid out to contain the pattern of the river prevailing at that time and no significant attempts were made to shorten the length of the floodway. Since that time the lateral movement of the channel has been limited by the floodway levees. Formation of sediment bars in the channel during low flow periods, and in particular on the recession of flood flows, together with rapid growth of water consuming trees and vegetation materially effect the channel configuration within the levees and are usually adverse to channel stabilization.

As pointed out previously, the channel of the Rio Grande has no banks. The levees which roughly parallel the channel act as banks and are subject to attacks by the river. Severe scour occurring during floods of long duration, causes relatively narrow deep channels to form which may shift toward the levees. Depth of these channels may reach 10 ft to 20 ft, even for moderate

TABLE 1.—FLOWS[a] WITHIN THE COCHITI-RIO PUERCO

Season (1)	Maximum flow (2)	Minimum flow (3)	100 yr. Frequency (4)	Approximate median (5)
Spring	25,400	0	36,500	1,000
Summer	27,300	0	42,000	1,000

[a] See flows in cubic feet per second

spring flood flows. The scouring effects of summer flood flows are less severe. When the levees are reached, failure by undermining and slumping into the scoured channel becomes imminent. The rate of shifting of the channels toward the levees is variable; 25 ft to 50 ft per day for moderate spring floods is not uncommon. Stabilization of the channel is required to prevent levee failure. Further, there is a pressing need for water salvage by reduction of channel area exposed to evaporation, better drainage, and sediment transport capacity which a stabilized channel will promote. A large volume of water has been lost by evaporation and transpiration in the channel of the Rio Grande. Water salvage is the term commonly used where these losses are reduced. There are no hydroelectric developments on the main stem of the river above Elephant Butte Dam.

FACTORS INFLUENCING CHOICE OF CHANNEL STABILIZATION WORKS

The principal factors influencing the choice of stabilization works were the streambed material, the relatively large sediment content of flood flows, and

the observed rapid growth of vegetation on the sediment beds within the floodway.

Because the streambed material is easily eroded, it was essential that any system of channel stabilization be capable of functioning efficiently under adverse conditions of channel scour. Therefore, a flexible system was indicated. Deposition of sediment and resulting rapid growth of vegetation indicated the desirability of encouraging depositon and growths in areas where the river channel was close to the levee. This would provide some protection to the levees and, at the same time, promote water salvage by reducing channel areas exposed to evaporation.

The magnitude of flood flows was also a factor. Engineering and economic considerations dictated the control of major flood flows and sediment by means of dams in conjunction with the floodway, and the type of stabilization works selected influenced the degree of control to be provided by the dams. Also, it was found that degradation of the channel, in the interest of flood control, major drainage, and water salvage could not be effectively accomplished by stabilization alone. Location of the dams is also shown on Fig. 1. Jemez Dam has been completed; Abiquiu Dam is under construction (in 1961) and the Cochiti and Galisteo Dams are in the pre-construction planning stage. When construction of the dams is completed, flood flows will be regulated as indicated in Table 2, based on available records at Bernalillo.

TABLE 2[a,b]

Season	Maximum flow	Minimum flow	100 yr. frequency
Spring	5,000	0	7,500
Summer	5,000	0	7,800

[a] See flows in cubic feet per second
[b] Median flow not available

From the foregoing, it is apparent that the principal functions of stabilization works after the construction of the dams will be: (1) to maintain the improved channel and protect the levees against reservoir releases and infrequent floods larger than those of record, which cannot be completely controlled by the dams; and (2) to effect water salvage and major drainage through channel degradation. The floodway is designed for floods of 20,000 cfs except at Albuquerque, where the summer flood capacity is 42,000 cfs.

TYPES OF WORKS EMPLOYED

The first significant attempts at stabilization of the channel of the river were made by the Middle Rio Grande Conservancy District during construction of the floodway. Emergency assistance at various times during and after major floods has been provided by the Corps of Engineers. These works were confined to levee protection in some of the more critical areas which had been repeatedly attacked during floods. The early protective works consisted of permeable jetties projecting from the levees toward the channel. Jetties were

constructed of wood piling placed in single or double rows and faced with woven wire. Some jetties were filled with brush and rock. The single row jetties were not very effective and were soon abandoned. The double row types were more effective; however, experience during moderate floods indicated serious deficiencies, the principal one being the inability to provide protection under conditions of significant channel scour. Since the piling was rigid, channel scour frequently undermined the structure and permitted the bulk of the flow to pass under the jetties with little velocity and energy reduction. Although ample sources of riprap were available, the depth of channel scour made protection by riprap economically infeasible.

In 1950, the Albuquerque District began large scale use of the Kellner jetty system on the Rio Grande in its emergency flood control work. This was done primarily because of the aforementioned deficiencies of the pile jetties and the rising costs of the pile type of channel stabilization work. The Kellner jetty system had been used by the Santa Fe Railroad in maintenance of way along the Rio Grande and Arkansas rivers for many years with such success that it has largely superseded other types in use by the railroad. The Kansas Highway Department has successfully utilized this system in protection of highways and bridges. The system was developed by the Kellner Jetties Company of Topeka, Kansas, in the early 1920's. H. F. Kellner started his experiments on a small stream near Topeka. The early unit consisted of willow poles tied together at mid-point and laced with wire. Experimentation with a variety of shapes and materials led to the present design. The system was formerly covered by patents which have now expired.

The Kellner Jetty System is well adapted to silt-laden rivers which are subject to considerable channel scour during periods of high river flow. The system is permeable, extremely flexible and readily conforms to channel scour. It permits sediment-laden water to penetrate the jetty where velocity and energy are reduced. As a result, sediment is deposited behind the jetty thereby building up an area or bank. Vegetation grows rapidly on the built-up areas and in a short time a secondary bank is established which not only provides protection but also reduces channel area exposed to evaporation. This promotes water salvage, an important consideration in semi-arid regions, The system is particularly adapted to the protection of levees, river banks, and bridge abutments.

The single unit of the system is called a "jack." Fig. 2 illustrates the standard unit. The unit consists of three 4 in.-by-4 in.-by $\frac{1}{4}$ in. steel angles placed at right angles to each other and bolted together in the center forming planes which are laced with wire. A lighter unit is also used, the only variation being substitution of 3 in.-by 3in-by-$\frac{1}{4}$ in. angles. Cables are usually 3/4 in.

All units are spaced at 12.5 ft centers and are connected together with wire rope to form jetty lines which in turn are connected together to form fetty fields. On the Rio Grande, jetty fields are used exclusively. The layout of a jetty field involves two types of lines of jetties. These are called diversion lines and tie back or retard lines. The diversion lines, usually two in number, and 12.5 ft center to center, are placed roughly parallel to the levee and along the desired location of the channel. The retard lines extend from the diversion lines back to the levees. They are usually placed on an angle of roughly 45° to 67.5° to the diversion line at the required spacing. At the major bends of the channel this spacing varies from 125 ft to 250 ft. Single diversion lines and wider spacing of retard lines (500 ft) are utilized in less critical areas,

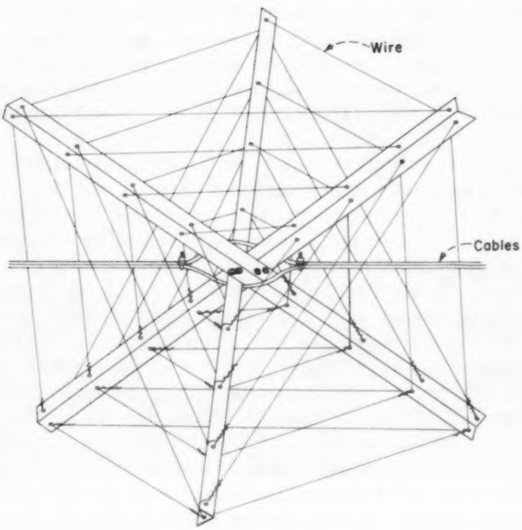

FIG. 2.—STANDARD UNIT

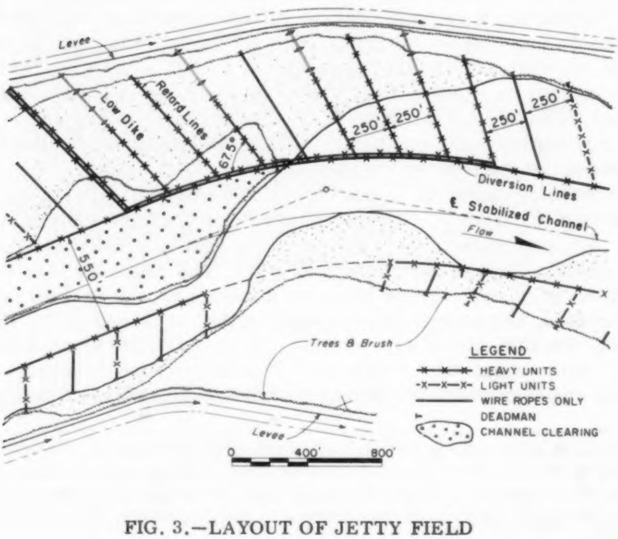

FIG. 3.—LAYOUT OF JETTY FIELD

however, wire ropes are provided between retard lines where the 500 ft spacing is used for intermediate anchorage.

When retard lines are long, low earth dikes may be substituted in alternate lines for a portion of the length close to the levee. Where brush and trees are heavy, retard lines may be terminated at trees. Anchorage is provided for retard lines by means of deadmen at levees or dikes or by anchorage to trees where lines terminate in trees and brush. Deadmen are usually standard creosoted railroad ties. Connecting wire rope is sized as follows: Double diversion lines, two-3/4 in. each line; corresponding retard lines two-3/4 in. Single diversion lines two-9/16 in.; retard line two-1/2 in. with light jacks.

Fig. 3 illustrates a typical reach of the floodway showing the layout of jetty fields with respect to the stabilized channel and the levees. Adequate design is important. Conditions prevailing on the Rio Grande dictated the layout shown in Fig. 2. Other areas may require jetty fields of different density. General principles usually followed in the layout of a jetty field and design curves have been developed.[2,3] The Corps of Engineers' report[2] summarizes the experience of the Albuquerque District and covers some of the installations of other agencies in the use of the Kellner jetty system at small isolated locations prior to 1953. The latter paper describes the results of model studies to determine effects that might be expected in typical jetty field layouts and on the experimental channelization and model of the Rio Grande in the Casa Colorado area, which is located near the lower end of the Cochiti-Rio Puerco unit of the floodway. Design curves are presented in the paper that can be used with judgment in the layout of a jetty field. This experimental work was valuable in determining the basic layout for the jetty fields on the Rio Grande.

Construction of a small jetty field is very simple. A crew of approximately twelve common laborers and a construction supervisor can assemble and place from 16 jacks to 25 jacks per day. Materials are delivered to the site by trucks at convenient locations. The jacks are assembled with hand tools and are carried to position in the jetty field by four or five men. If the work is being done in water, they are rafted into place. Usually about 16 units are assembled and positioned at a time. The cables are threaded through the units and clamped. Natural vegetation is disturbed as little as possible.

Where larger fields are used, as on the Rio Grande, a mechanized assembly and placement procedure is being used. One highly satisfactory plant consists of a tractor mounted crane drawing a flatbed trailer, approximately 50 ft long and 10 ft wide. The forward half of the trailer contains material racks for angles, wire rope, and a space for assembling the angles. The rear half contains space for lacing two jacks. The crane handles the stack of angles and the wire rope spools from the material trucks to the racks. Lacing wire, in bundles, and precut to required lengths, is placed in the lacing area.

To start, the line angles are lifted by the crane to the assembling position and bolted with pneumatic wrenches. The wire rope is passed around the bare angles and clamped. The trailer moves forward about 12.5 ft placing the

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2 "Report on Measures for Bank Protection, Use of Kellner Jetties on Alluvial Streams," Corps of Engrs., U.S. Army, Office of the Dist. Engr., Albuquerque, N. Mex., June, 1953.

3 "Use of Steel Jetties for Bank Protection and Stabilization in Rivers," by Enos J. Carlson and Philip F. Enger, Hydr. Engrs., Commr's. Office, Bur. of Reclammation, Denver, Colo.

jack in the first lacing position where it is partially laced. The trailer moves forward another 12.5 ft which places the jacks in the last lacing position where lacing is completed. Meanwhile, two more jacks have entered the assembly line. Thus, with each forward movement of 12.5 ft, a completed jack drops off

FIG. 4.—CONSTRUCTION PLANT

the end of the trailer in proper position with connecting wire ropes properly clamped.

Fig. 4 illustrates the plant, devised by The J. R. Cantrall Corporation, El Monte, California, prime contractor for some of the jetty works. This operation requires a supervisor and a crew of sixteen men. The capacity is about 125 jacks per day under average conditions. The plant can operate in

water up to about 8 in. deep. When greater depths are encountered, the area is dewatered by filling in or by temporary diversion; or if dewatering is infeasible, the hand method that was referred to previously is utilized. Two such plants are normally used. Fig. 5 shows some of the completed jetty fields.

FIG. 5.—COMPLETED JETTY FIELDS

All of the jetty installations accomplished (as of 1961) have been by contract. Jetties are usually bid on a unit basis which includes all items of installation including wire rope, deadmen, cable clamps, and so on. Recent

contract costs of the units have averaged about $45.00 for the heavy units and about $37.00 for the light units. The total contract cost of the channel stabilization for the entire 105 mile Cochiti Rio Puerco unit of the floodway is estimated to be about $6,000,000 or about $57,000 per mile. About $250,000 of the total contract amount will be expended in channel clearing and necessary pilot channels through bars. It must be borne in mind that these costs are for both sides of the channel. The project is scheduled for completion in 1962. Cost comparison with the wooden pile types previously used are not available because these types were abandoned due to inadequate performance.

EFFECTIVENESS OF STRUCTURES

The earliest uses of the Kellner jetty system in the Middle Valley area were by the Santa Fe Railroad in protection of railway embankments beginning about 1936. This work was on a limited scale with satisfactory results. The first large scale use of the system was by the Corps of Engineers in the Middle Valley in 1953. Seven jetty fields were constructed under emergency authority at isolated locations where the levees were exposed to attack by the river. In all, about 5,600 units were installed. The designs of the individual fields were similar to the double diversion line field shown in Fig. 2, except that the retard lines were spaced at 125 ft and were considerably shorter. Figs. 6 illustrate "before" and "after" conditions at the installation near Bernalillo, New Mexico. Fig. 6(a) was taken in December 1952 looking downstream prior to installation of bank protection works. Fig. 6(b) was taken in August 1953 at the same location after installation of the Kellner jetty field in June 1953. Fig. 6(c) was taken in September 1955 (vegetation has grown in the jetty field). Fig. 6(d) was taken in April 1958.

Construction of the stabilized channel within the Cochiti to Rio Puerco reach of the floodway began in 1954 within the Albuquerque unit. By 1956 about 17,000 additional units had been installed. From 1956 to 1958 no work was accomplished. Work was recommenced in 1958 and approximately 50,000 more units have been installed. The entire work will be completed in 1962 at which time a total of approximately 115,000 units will have been installed.

Table 3 indicates the maximum peak flows to which the jetty fields have been exposed since 1953. The period represented by Table 3 is considered to be a period of sub-normal flow and conditions favorable to stabilization of the jetty fields have prevailed. The flood flows have been moderate. Sedimentation and growth of vegetation within the jetty field which usually occurs in this area within 2 yr to 3 yr have been adequate. There have been no failures in the jetty fields which have been installed.

Repair and maintenance of the jetty fields are usually accomplished during period of low flow by installation of additional lines of jacks in areas that have been subjected to extreme scour conditions. In such cases the wire ropes connecting the new jack line is secured to the original line upstream and downstream of the scoured area and the line of jacks placed adjacent and parallel to the original line. If the original line has sunk considerably the line of jacks may be placed on top of the original line.

In areas that have not been protected by jetty fields, it has been necessary during periods of flood fighting to utilize the jetties to prevent loss of sections of the levee exposed to channel scour. In this case lines of jacks are assembled on top of the threatened levee. When the assembly is complete with suitable

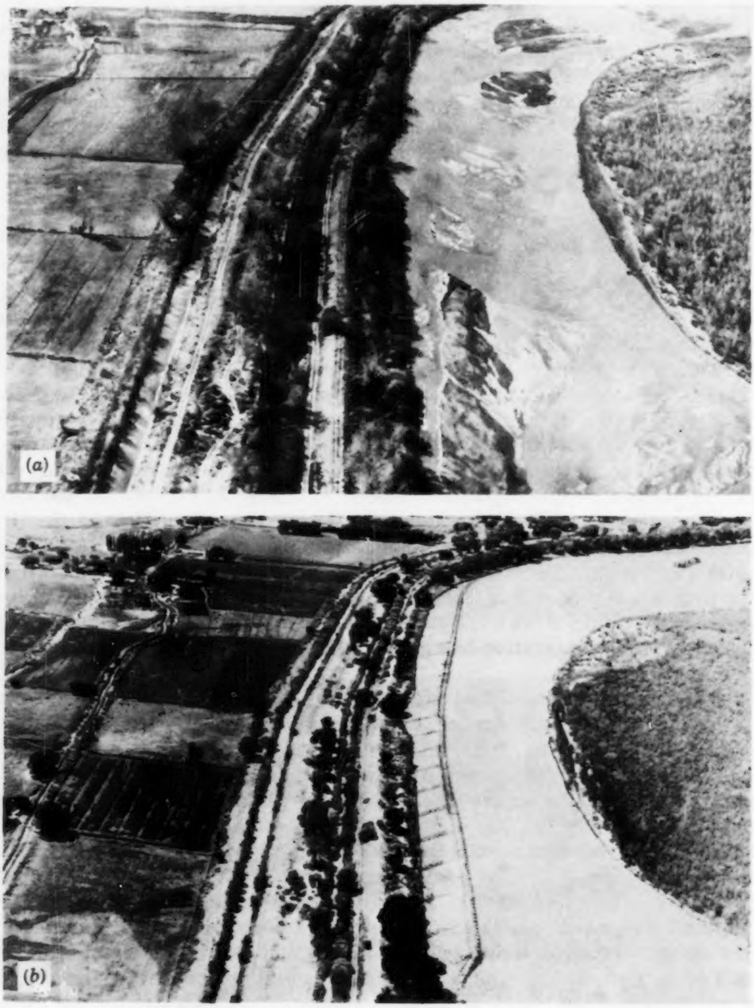
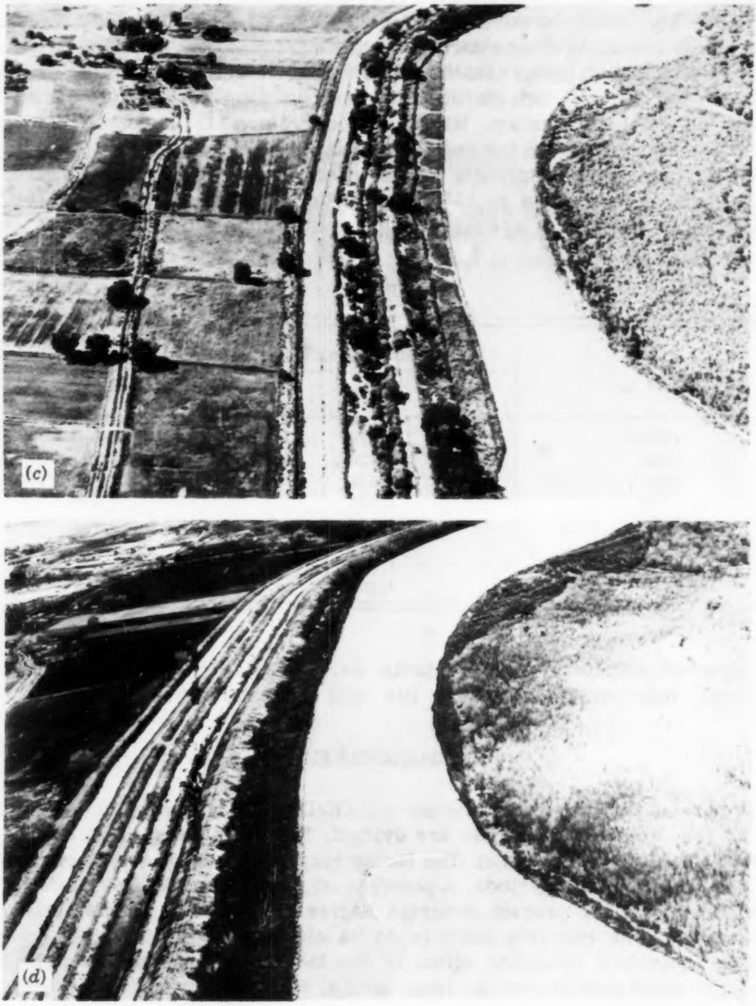


FIG. 6.—EMERGENCY FLOOD CONTROL WORK—

slack anchor lines, it is tipped over into the scoured area at the toe of the levee. A second line may be necessary where scour has been severe.

Maintenance costs on properly designed jetty fields are moderate. No maintenance has been required on the 5,600 units installed in 1953, nor on the additional 17,000 units installed prior to 1956. As the project is still under construction, it has been necessary to estimate the future maintenance costs. It is estimated that the entire 105 miles of channel stabilization can be main-



RIO GRANDE NEAR BERNALILLO

tained over a 50 yr period at a cost of approximately $130,000.00 per year, including channel clearing and replacement of 50% of the jetty units during the 50 yr period. This amounts to approximately $1,250.00 per mile per year. The estimate is predicated on the assumption that all the authorized flood control dams will be in operation within a reasonable period after completion of the channel stabilization.

The effective life of a Kellner jetty field should be at least 25 yr under conditions prevailing in semi-arid regions such as the Rio Grande Valley. In more humid areas, or where corrosive water is found, the effective life would be shorter. The previously mentioned emergency installations by the Corps of Engineers on the Rio Grande in 1953 are intact after 7 yr of service with no visible sign of deterioration. While some corrosion has no doubt taken place in the portions buried in the sediment beds, severe corrosion of some of the buried portion would not greatly impair effectiveness. The works constructed by the Santa Fe Railroad in 1936 on the Rio Galisteo, a tributary of the Rio Grande above Bernalillo, are fully effective after approximately 25 yr of serv-

TABLE 3

YEAR	Maximum peak flows in cubic feet per second	
	Spring	Summer
1953	2,880	9,100
1954	1,920	4,280
1955	2,580	13,300
1956	1,260	7,300
1957	7,140	13,900
1958	11,800	5,700
1959	1,020	2,500
1960	4,800	

ice, although additional rows of jacks were installed in 1952. It is probable that with this replacement, the life will be extended an additional 25 yr.

RECOMMENDATIONS

No serious deficiencies in the use of the Kellner jetty system in channelization of the Middle Rio Grande are evident. The jacks being made entirely of steel are subject to corrosion. The lacing may be broken by debris carried by the river during flood periods. A possible improvement might be made by the substitution of light precast concrete angles for steel and the elimination of lacing wire. The concrete units could be sized and spaced in lines so as to provide equivalent retarding effect of the laced jack lines and would be less subject to corrosion or damage from debris. While this would not be economical for conditions prevailing on the Rio Grande, it might be elsewhere.

Large scale use of the Kellner jetty system for channel stabilization on other rivers should be preceded by construction of a test reach and, if possible, investigation in the laboratory to predict performance at the range of discharge expected.

CONCLUSIONS

Data, collected from experience, on stabilization of the channel of the Middle Rio Grande River in New Mexico have been presented and analyzed herein.

Channel stabilization of the Middle Rio Grande is being accomplished effectively and economically by the use of the Kellner jetty system.

This system is particularly adapted to the wide shallow rivers of the Southwest, such as the Rio Grande, in which sediment content is high and the river bed is subject to considerable scour during floods.

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Journal of the

WATERWAYS AND HARBORS DIVISION

Proceedings of the American Society of Civil Engineers

BANK AND LEVEE STABILIZATION, LOWER COLORADO RIVER

By John S. McEwan[1]

SYNOPSIS

The Bureau of Reclamation, United States Dept. of Interior has been involved in the problems of levee protection, river bank stabilization, and sedimentation on the Lower Colorado River for several years. The procedures used for providing levee protection under the early conditions of no storage dams on the river generally involved large quantities of quarry-run stone placed on the levee by means of railroad cars. Conditions in 1961, with a high degree of flood protection provided by Hoover, Davis, and Parker Dams call for different methods.

Channelization activities on the Lower Colorado River still provide levee protection to surrounding land areas; however, greater stress is given to bank stabilization along the normal stage channel banks in order to maintain the channel in its designed location and alinement and to control the movement of sediment from the alluvial banks. The methods being used by the Bureau of Reclamation are the result of 50 yr of experience.

Innovations in the methods of placing riprap stone for bank stabilization purposes have resulted in the development of fast and economical procedures

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that have proven entirely satisfactory when used on the non-cohesive, alluvial banks typical of the Lower Colorado River.

INTRODUCTION

The levee revetment and bank stabilization activities of the Bureau of Reclamation, United States Dept. of Interior (USBR) along the Lower Colorado River are a part of the Bureau's assigned functions under the Colorado River Front Work and Levee System. This project which began as a small flood control operation at Yuma, Ariz., has grown to a designated area of responsibility extending along the channel of the Colorado River from Lee Ferry, near the Utah-Arizona line, to the International Boundary between the United States and Mexico.

Colorado River Front Work and Levee System has as its basic authority Public Law 469-79th Congress, 2nd Session, which was approved June 28, 1946. This act amended the earlier acts whereby the USBR performed flood-protection work in limited areas and charged USBR with the complete responsibility for operation and maintenance of the river system over the entire length of the Lower Colorado River. The following statement is quoted from the act that defines the scope of the Bureau's activities:

> "(A) Operating and maintaining the Colorado River Front Work and Levee System in Arizona, Nevada, and California; (B) constructing, improving, extending, operating, and maintaining protection and drainage works and systems along the Colorado; (C) controlling said river, and improving, modifying, straightening, and rectifying the channel thereof; and (D) conducting investigations and studies in connection therewith: . . ."

RIVER CHARACTERISTICS

The levee revetment and bank stabilization efforts of the USBR on the Colorado River have extended into two separate periods so far as the characteristics of the river are concerned. Prior to the construction of the storage dams on the river, floods of considerable magnitude were to be expected every spring. Records indicate that the maximum recorded flood at Yuma, Ariz., reached 250,000 cfs, and the minimum flow was 18 cfs.

Present (1961) river characteristics show that the flow of the river has been smoothed out and regulated to a high degree. Maximum regulated flows under present operating conditions are held to the maximum hydraulic capacity of the Parker Powerplant, which is about 23,000 cfs. Naturally, releases higher than this are possible, but would not be resorted to except under flood conditions. Minimum regulated flow is fixed by the minmum flow possible through one turbine unit at Parker Dam of about 2,000 cfs. Average flow in the Davis Dam to Palo Verde Diversion portion of the river is usually taken as 15,000 cfs, and between the Palo Verde Diversion Dam and Imperial Dam it is customarily taken at 10,000 cfs. These figures are not true mathematical averages, but are the usually accepted working relationships.

The overall slope of the river channel from Davis Dam to the International Boundary is 1.4 ft per mile. Considerable variation exists within limited reaches; however, these do not range to more than about 3.0 ft per mile as a minimum.

The material comprising the bed and banks of the channel is classed generally as fine sand. In some areas the tributary washes have contributed large quantities of gravel to the river channel which serves to armor the bed of the channel but leaves the banks very susceptible to large scale bank erosion. Bed samples taken above Parker Dam show the D-50 particle size to range from 0.50 mm to 0.20 mm. Below Parker Dam, the D-50 size ranges from 0.35 mm to 0.20 mm. Indications are that particularly in the reaches below the storage dams, the bed material is shifting toward the coarser fractions.

YUMA LEVEES

Early efforts of the USBR to control the Colorado River were made prior to the construction of the storage dams on the Lower Colorado River. Essentially, these efforts took the form of a levee system designed to protect the Yuma Project.

The experience of the early reclamation engineers in the Yuma area dictated the use of heavy revetment or riprap from one and of the levee to the other. The riprap materials were obtained as quarry-run stone from sources located as near to the levees as was physically possible. At the times railroad cars were the only available means of hauling the large volumes of riprap materials that were required for the levee. No real effort was made to select the material by size; however, due to the problems involved in handling the material it is unlikely that anything over 1/2 cu yd size was used.

The history of the performance of the levees reveals that on very few occasions were the levees overtopped. In practically all cases the failures that developed were due to the undermining of the levee revetment by direct attack of the river current and the subsequent collapse of the levee embankment. It should be stated here that the conditions of flow that were being fought involved the unrestrained spring runoff of the Colorado River, with ever-present threat of a flash flood originating on the Gila River. Flows ranging from 2,000 cfs or 3,000 cfs to over 100,000 cfs were to be expected almost any year. The duration of the flood stages could be anything from a few hours to 3 weeks or 4 weeks. Under these conditions, it is surprising that the failures were not more numerous than they were.

Subsequent to the completion of Hoover Dam, little if any maintenance work was done on the Yuma Levees, and no floods were experienced during the period of 1935 to 1950. With the authorization of the Mexican diversion structure at the Morelos site below Yuma, the flood hazard in the Yuma area was again brought under study. Recomputation of the flood hazard at this time resulted in the rehabilitation and raising of the levees with funds provided by the United States and Mexico.

In the reconstruction of the Yuma Levees, the levee section was increased and the grade raised. Revetment consisting of heavy riprap was placed at critical locations where the possibility of an attack by the river appeared to be most likely. The riprap used was limited to a maximum size of 1/2 cu yd

and was graded on down to rock spalls not less than 1/10 cu ft in volume. The specifications allowed the placement of the riprap to be made by dumping the riprap on the shoulder of the levee and pushing it over the levee slope with a dozer. Two thicknesses were specified which required 5 ft or 10 ft of riprap to be placed on the levee face, measured normal to the slope. The 10-ft thickness was placed where the river was particularly close to the levee and presented an immediate hazard to the safety of the levee embankment.

Consideration of the changed conditions as regards the flood hazard and the duration of any possible flood influenced the decision not to provide complete revetment of the levees during the rehabilitation period. Whereas under the original conditions prior to Hoover Dam, a flood of high magnitude and long duration was frequent, under the conditions imposed by the construction of the storage dams on the river, the most probable type of flood was thought to be a moderately high magnitude flood of rather short duration and rare frequency, induced by a high-intensity storm centered over the Lower Colorado River Basin. Complete levee revetment, therefore, was not included as part of the reconstruction of the Yuma Levees, but provisions were made to periodically extend the riprap on the levees during the operation and maintenance phase.

On one occasion, the 10-ft thickness of riprap was called on to withstand a direct impingement of the force of the river current against the levee. It should be explained that at no time during this attack was the river at flood stage. The river merely shifted its channel and brought the force of the current against the newly constructed levee. Fortunately for the safety of the levee, the 10-ft thickness of heavy riprap was sufficient to hold against the force of the current, although a considerable amount of the levee revetment was lost by undercutting and sloughing. A minor channelization project upstream from the point of attack was necessary to relieve the river pressure on the levee.

CHANNELIZATION THROUGH MOHAVE VALLEY

In 1949, the USBR began the channelization of the Colorado River near Needles, Calif. This presently completed project involved the channelization by dredging methods of the 32 miles of river between Big Bend and Topock, Ariz. As a result of this project, the river has been confined to a rectified channel with an average width of 500 ft and on a grade of approximately 1-1/2 ft per mile. Degradation at the upper end of this channel will eventually reduce the gradient to about 1-1/4 ft per mile.

The rectified channel was designed to carry an average flow of 15,000 cfs with daily fuluctuations due to the operation of the powerplant at Davis Dam ranging from 4,000 cfs to about 18,000 cfs. Levees were provided to contain a flood of 50,000 cfs in the upper end of the channel, and 70,000 cfs in the lower portion.

The first dredging activity began at Needles, Calif., and proceeded downstream to Topock, Ariz. The second phase of the work consisted of the channelization of the river upstream to the lower end of Big Bend, some 12 miles downstream from Davis Dam.

During the first phase of the work emergency conditions at Needles required the completion of a free-flooding channel at the earliest possible time. Therefore, the method of operation of the dredge was simply to cut a pilot

channel of sufficient width to immediately capture the flow of the river. The resulting channel was narrower than the designed width; however, it was contemplated that the river would widen its channel by erosion of the banks until the design width was reached. To prevent the formation of over-wide sections, it was planned to provide bank stabilization measures as the need developed.

Various methods of bank stabilization were tried with something less than 100% success. One of the early methods attempted involved the use of permeable structures designed to cause a deposition of river-borne sediment in a designated area. These structures consisted of railroad rails jetted vertically into the bed of the river at 3-ft to 5-ft intervals and hung with 6-in. mesh wire fencing. The action of the structure was much the same as a snow fence with an induced deposition taking place behind the structure. Initially, the design appeared to hold considerable promise; however, it soon became evident that in order for the structure to work satisfactorily, the river had to be carrying a fairly high concentration of sediment. Under the conditions of velocity of flow being experienced, which was from 3 fps to 6 fps, it was found that a concentration of sediment in the river approaching 700 ppm was required to make the desired deposition take place. With the average concentration running at about 300 ppm or less, the small amount of turbulence set up by the structure induced erosion rather than deposition.

Kelner-type jetties were tried with about the same results. Eventually, all the metal structures were replaced with riprap groins or hard points spaced along the erosion areas. These have been successful in stopping the widening of the channel beyond design limits; however, they have the disadvantage of giving the bankline an unsightly scalloped effect.

In the second phase of the Mohave Valley channelization work, a considerably different approach to the matter of bank stabilization was made. With the dredge progressing upstream from Needles toward Big Bend, the decision was made to dredge very nearly to the full design width and to place bank stabilization materials directly on the concave side of all bends. Because the channel design consisted of a series of curves of about 10,000 ft radius, the length of bank stabilization contemplated was approximately one-half of the total length of bankline.

It was recognized that there would be difficulties in placing riprap on the steeply cut banks with no access roads available to work from. An initial attempt was made to place the riprap from a barge using a small dozer to push the rock material over the edge of the steel deck. The method was abandoned after a short trial when it became evident that the dozer could not place the riprap on the bank above the water line.

The next method tried was to end dump the riprap from dump trucks directly on the cut bank. This required an access road to be built right at the edge of the bank so that the dump trucks could negotiate over the soft sand and silt that composed the banks. If the access road was built any distance ahead of the bank stabilization work, it was often lost to the widening of the river and, if it was built back from the bank, the trucks could not maneuver in the soft sand between the road and the bank.

At this stage of the work, a new approach to the placement problems that had slowed the progress of the bank stabilization work was attempted. Because the bank material was entirely non-cohesive sand and silt, it appeared possible to place the riprap material on the ground surface in the form of a windrow

FIG. 1.—PLACING WINDROW ON DRY LAND

FIG. 2.—ACTION OF RIVER WITHIN WINDROWS

at some distance back from the cut bank and let the river cut the bank back until the windrow was reached. As the river cut the bank away, the riprap material from the windrow began to drop down the cut bank, the riprap spread itself over the exposed bank, and the cutting action was gradually brought to a halt.

This method of riprap placement has the advantage that the final bankline can be selected in advance and the riprap placed accurately on the selected line. The quantity of riprap used in the windrow can be adjusted to fit the local conditions. In the Mohave Valley work, the rate of riprap placed has averaged about 2 cu yd per linear foot of bank. Maximum size rock used was 1/4 cu yd. The resulting bankline is a neat appearing line that tends to stablize on the surveyed line laid out for the original windrow. Fig. 1 shows the operation of placing the windrow on dry land, and Fig. 2 shows the action of the river within the windrows. It should be noted in the photograph that irregular portions of bank remain within the design bankline. These later eroded away, and the river stopped cutting at the riprap windrow.

The plan that riprap would be used only on the concave side of the channel has not been exactly adhered to. In many cases, it has been necessary to stabilize both banks. Where this has been done, the banklines are parallel and a good channel cross section results.

The experience in Mohave Valley with placement of riprap in windrows has been entirely successful and has resulted in advantages that were not entirely anticipated. Replacement or addition to the original riprap has been necessary in only a few instances; however, the original access road is available for use and replacement of riprap can be made without access difficulties. Over the 32 miles of channelized river in Mohave Valley, it has been necessary to use a total of 106,500 cu yd of quarried rock to accomplish the initial stabilization. Future maintenance will require an additional amount to be placed; however, it is not believed that this will require any large amount. Fig. 3 shows the completed channelization project with bank stabilization in place and functioning. The city of Needles, Calif., is in the foreground.

FUTURE WORK

It is anticipated that the USBR will undertake the channelization of the Colorado River through Palo Verde and Cibola Valleys. This new project is logically divided into two sections in which different methods of channelization will be used. The upper end of the project through Palo Verde Valley is within the degradation area induced by Parker Dam, the old Palo Verde Weir, and its successor, the Palo Verde Diversion Dam. In this part of the project, the plan is to channelize the river by means of bank stabilization works without the benefit of dredging. The use of training structures and bank protection works will be quite extensive. It is believed that in an area of degraded channel, these methods will be adequate to establish the river in a definite location with satisfactory alinement.

In the Cibola Valley portion of the channelization project the river is in an aggraded section, and in this location it is believed that it is necessary to excavate a new channel by dredging methods before it would be possible to put the channel into a satisfactorily rectified condition. The bank protection plan for this part of the project includes riprap stabilization on all of the concave banks and most of the convex banks. Placement of this riprap will, when-

ever practicable, be scheduled ahead of dredging. The riprap will be deposited in windrows along the designated channel bankline so that as the bank is finally eroded to line, the rock will drop into place in a smooth blanket. Gradation of the rock will be that obtained in quarry-run stone with the maximum size limited to 1/4 cu yd.

The combined effect of the Palo Verde Valley channelization by bank control methods and the Cibola Valley channelization by dredging methods will be to confine the Colorado River in a definite channel and restrict the sediment movement in the river to that sediment which enters the reach from above, plus

FIG. 3.—COMPLETED CHANNELIZATION PROJECT

the limited sediment available in the bed of the channel. Present measurements of sediment transport indicate that sediment passing the lower end of Cibola Valley under conditions that are anticipated for future years will average about 2,750,000 tons per yr. Estimates prepared in connection with the authorizing report for the Palo Verde-Cibola work place the sediment transport under completed project conditions at approximately 1,800,000 tons per yr. This represents a reduction in sediment passing the lower end of Cibola Valley of 950,000 tons per yr. Such reduction of sediment transport is due, in a large

measure, to the denial of a source of sediment to the river by the bank stabilization features of the project.

Long-term future work of a similar nature is under consideration for the section of the river between Headgate Rock Dam near Parker, Ariz., and the Palo Verde Diversion Dam. Because this area is in the degradation area below Parker Dam, methods of channel control similar to those proposed in the Palo Verde Valley would most likely be used.

Other problem areas exist along the river where sediment movement can be controlled by bank protection and proper channel design. Eventually the provision of a low-water channel may become necessary in the section of river from Imperial Dam to the international boundary.

These projects may be several years in the future but they are all parts of the general overall plan of the USBR to bring the Lower Colorado River under full control and, by doing so, to reduce the bank erosion, sediment movement, and water losses to a practical minimum.

CONCLUSIONS

Fifty years of levee revetment and bank stabilization activities of the USBR on the Lower Colorado River have developed new and economical procedures for placing riprap materials. These methods are particularly suited to the stabilization of the raw banks associated with a channelization project.

By replacing the riprap stone in a windrow on dry land and allowing the river to cut back to the windrow, a neat appearing and easily maintained bankline results. The use of this method requires that the soil composing the banks be non-cohesive, alluvial material.

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NON-LINEAR TIDAL FLOWS AND ELECTRIC ANALOGS

By James A. Harder,[1] M. ASCE and Frank D. Masch, Jr.,[2] A. M. ASCE

SYNOPSIS

To illustrate the convenience with which electrical analogs may be used to investigate the hydraulic characteristics of a non-linear tidal system, an electrical model of the proposed sea-level Panama Canal was constructed and measurements of tidal elevations and discharges were made at the mid-point and the two ends of the canal. Mean velocities were computed and some comparisons were made with results of hydraulic model studies. The necessity for utilization of non-linear elements in the analog when necessary to obtain accurate descriptions of the wave forms is emphasized.

INTRODUCTION

The propagation of tidal waves in estuaries and channels is one of the more difficult problems encountered in hydraulics. This complexity stems from the non-linear nature of the differential equations which describe the unsteady flow in the tidal system and the often complex tides which must be used as boundary

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conditions. Some analytical solutions have been obtained by linearizing the equations of motion and in many instances, these solutions have proven to be entirely satisfactory. In some cases, however, simplifications have been carried to the extent that the nature of the problem was lost.

Physically, the problem in a tidal system is one of a complex wave moving through a channel. The shape of the wave is continuously changing because the various components of the wave travel at slightly different speeds. The motion may be further complicated if reflections and resonant amplifications occur within the tidal estuary.

To obtain accurate descriptions of this complex motion, it is generally necessary to solve the equations by numerical methods. In the event it is desirable to investigate the influence of the different variables on the tidal flow, the usual procedure is to build small scale hydraulic or electrical models. Of the various numerical and graphical solutions proposed, the method of characteristics which is based on the work of J. Massau,[3,4] appears to be the most general, and it is a method which lends itself to computation by digital computers. Although satisfactory results can often be obtained from this calculation, it is usually not practical to determine solution trends because of either the lengthy hand calculations or the expensive operations of a digital computer.

As a result of the limitations imposed by computational methods, extensive use has been made of small scale hydraulic models. Many of the complications of the tidal motion are automatically taken care of by the model, and it is relatively simple to vary the boundary flow conditions. However, the construction and verification of hydraulic models as well as their operation and general upkeep is expensive in terms of both time and money. Furthermore, changes in physical configuration are not easily attained because they almost always involve modifications to the model itself.

In many ways the electrical analog is similar to the hydraulic model. Some non-linear aspects of the flow are handled within the analog, and the boundary flow conditions are easily varied. Although electrical analogs obviously cannot be used to study such problems as the scour and deposition of sediment, they are particularly useful in strictly hydraulic investigations. Solution trends are easily determined, and changes in the physical configuration of the prototype are usually represented by adjustments of the electrical elements in the analog. Due to a compressed time scale, the effect of a variance in the conditions at one point in the tidal system is known almost immediately at all other points. In general the reliability of an analog depends on the accuracy of the field data. With proper care in the selection of the analog elements, the results obtained from the electrical model will be as accurate as those obtained by other methods. Electrical analogs are relatively inexpensive and can be constructed and verified in a short period of time. The operation and maintenance costs are small in comparison to the hydraulic model.

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3 "Memoire sur l'integration graphique des equations aux derivees, partielles," by J. Massau, Annales de l'association des ingenieures sortis des Ecoles Speciales de Gand, Vol. 23, 1900, p. 95.

4 "L'integration graphique," by J. Massau, Annales de l'association des ingenieures sortis des Ecoles Speciales de Gand, Vol. 12, 1889, p. 435.

Notation.—The letter symbols adopted for use in this paper are defined where they first appear, and are arranged alphabetically, for convenience of reference, in the Appendix.

ESSENTIAL OF ELECTRICAL ANALOGS

The well-known equation for unsteady open channel flow is

$$\frac{1}{g\ A}\left[\frac{\partial Q}{\partial t} - 2\ \frac{Q\ b}{A}\ \frac{\partial y}{\partial t} + \frac{Q^2}{A^2}\ \frac{\partial A}{\partial x}\right] + \frac{\partial y}{\partial x} + \frac{|\,Q\,|\,Q}{C_z^2\ A^2\ R} = S_o \quad \ldots\ldots (1)$$

in which Q is the discharge, A the cross-sectional area, b denotes the surface width, y refers to the water depth, R is the hydraulic radius, S_o the bed slope, C_z the Chezy coefficient, x describes the distance along the channel, and t is the time. This equation assumes that for tidal flows, the wavelength is long in comparison to the wave height, pressure distributions are hydrostatic, vertical components of velocity are negligible, and there are no significant lateral flows. Briefly, the acceleration term in Eq. 1 consists of the reaction due to the time variation of discharge at a given cross section, the effect of a changing water surface elevation on the reaction given by $(\partial Q)/(\partial t)$, the reaction due to a change in velocity head induced by $(\partial Q)/(\partial t)$, and the effect of a change in velocity head due to a change in channel cross-section. The second and third parts of the acceleration are combined in the term, $-\ (2\ Q\ b/A)\ (\partial y/\partial t)$. The friction slope in Eq. 1 has been replaced by the conventional Chezy equation. Absolute value notation is necessary because the friction slope changes sign as the flow changes direction. If it is also assumed that the wave height is small in comparison to the water depth and that the channel width is essentially constant, Eq. 1 reduces to

$$\frac{1}{g\ A}\ \frac{\partial Q}{\partial t} + \frac{\partial y}{\partial x} + \frac{|\,Q\,|\,Q}{C_z^2\ A^2\ R} = S_o \quad \ldots\ldots\ldots\ldots (2)$$

The continuity equation written in terms of the discharge is

$$\frac{\partial Q}{\partial x} + b\ \frac{\partial y}{\partial t} = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

Complete derivations of the Eqs. 2 and 3 may be found elsewhere.[5] It should be noted that the preceding equations are written in terms of the discharge as there is no quantity in an electrical circuit that corresponds to a velocity.

The electrical equations for a transmission line are also included here without derivation. These equations are

$$L\ \frac{\partial i}{\partial \tau} + \frac{\partial e}{\partial \xi} + r\ i = E_o \quad \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

5 "The Theoretical Basis for Non-Linear Electric Analogs for Open Channel Flow," by James A. Harder, Univ. of California, Inst. of Engrg. Research, Series No. 107, Issue No. 1, June, 1957, pp. 13-15.

and

$$\frac{\partial i}{\partial \xi} + C \frac{\partial e}{\partial \tau} = 0 \quad \text{. (5)}$$

in which i is the current; L, C, r, and E_o are the inductance, capacitance, resistance, and applied potential per length of transmission line respectively; e denotes the voltage, ξ refers to the distance along the transmission line, and τ describes the time.

From a comparison of Eqs. 2 and 3 with Eqs. 4 and 5, the following correspondence of variables and parameters is noted:

Hydraulic Variables and Tidal System Parameters		Electrical Variables and Transmission Line Parameters	
Q	- discharge	i	- current
y	- water depth	e	- voltage
x	- distance	ξ	- distance
1/g A		L	- inductance per length
b	- surface width	C	- capacitance per length
t	- time	τ	- time

For a complete analogy between the hydraulic and electrical systems, there must also be a termwise correspondence between the equations, and the ratio between each of the corresponding terms must be constant. From this basic principle, the equations relating the electrical and hydraulic parameters can be shown to be

$$\frac{1}{\tau} \sqrt{L\ C} = \frac{1}{t\ C_o} \frac{x}{\xi} \quad \text{. . . . . (6)}$$

and

$$\sqrt{\frac{L}{C}} = \frac{1}{b\ C_o} \left[\frac{\frac{Q}{i}}{\frac{y}{e}} \right] \quad \text{. . . . . (7)}$$

In Eqs. 6 and 7, $\sqrt{g\ a/b}$ has been replaced by C_o the celerity of a shallow water wave. For a linearized-friction analog, the electrical resistance is related to the hydraulic friction by the equation

$$r = \frac{|\,Q\,|}{C_z^2\ A^2\ R} \left[\frac{\frac{Q}{i}}{\frac{y}{e}} \right] \frac{x}{\xi} \quad \text{. . . . . (8)}$$

Although the analogy between the variables in Eqs. 2, 3, 4, and 5 is quite evident, this precise correspondence does not exist if the effects of friction or finite wave amplitude are considered. In the electrical equations the resistance term is linear, and for finite wave amplitudes, the linearity of the equations is generally unaffected. Under similar conditions it is noted that the hydraulic equations become non-linear. To this extent most electrical analogs

used in the past have not completely described the true flow conditions in tidal systems. Before examining the application of the analog, it is well to consider the non-linear nature of the flow in a tidal canal.

Non-Linear Effects of Friction.—A comparison of Eqs. 2 and 4 indicates that the electrical resistance, r, must correspond to the hydraulic quantity, $|Q|/\left(C_z^2 A^2 R\right)$. If the current, i, is to be analogous to the discharge, Q, the value of the resistance must depend on the current. This requirement leaves three alternatives for the inclusion of friction in the analog.

As a first alternative, the friction may be linearized by setting the quantity, $|Q|/\left(C_z^2 A^2 R\right)$, equal to a constant such that the total energy dissipated over a tidal cycle is equal to that which would occur if the friction were non-linear. Although proper linearization of the friction correctly attenuates the wave motion, other non-linear effects such as the introduction of higher harmonics is lost. In deep channels, however, friction is small and although it may be the main source of higher harmonics, only few are generated and the use of linearized friction may be entirely satisfactory.

The second alternative for the inclusion of friction is to adjust the resistance in each section of the analog to correspond with the current flowing therein. This method allows the use of linear resistors, but it invovles a trial and error procedure which may become time consuming in large complicated tidal systems.

The third method that may be used to introduce friction into the analog is to replace the linear resistor by one which induces a voltage drop proportional to the square of the current. Such a unit is the Transistorized Square-Law Resistor developed by Hans A. Einstein, F. ASCE and Harder.[6] This resistor eliminates the need for successive adjustments and once set for the channel roughness will dissipate energy in the proper way and retain the higher harmonics generated by non-linear friction.

Non-Linear Effects of Changes in Water Depth.—The simplified expression given as Eq. 2 shows that changes in water depth will affect both the inertia term and the friction term. It can further be seen from the more general relation given in Eq. 1 that other terms are affected if the wave has a finite amplitude. Based on the work of Harder,[7] an analysis of these non-linear effects and their influence on the flow will be presented.

Generally speaking, tidal estuaries are much wider than they are deep. Consequently, a change in water depth of y' will change the area by an amount approximately equal to $b_o\, y'$, where b_o is the surface width at mean depth. In the following analysis, zero subscripts represent values at mean depth. In addition a change in water depth of y' will also change the hydraulic radius from R_o to $R_o + R'$. Assuming the surface width, b_o, remains a constant, and set-

6 "An Electric Analog Model of a Tidal Estuary," by H. A. Einstein and James A. Harder, Proceedings, ASCE, Vol. 85, No. WW 3, September, 1959, pp. 161-164.

7 "The Theoretical Basis for Non-Linear Electric Analogs for Open Channel Flow," by James A. Harder, Univ. of California, Inst. of Engrg. Research, Series No. 107, No. 1, June, 1957, pp. 34-40.

ting $y = y_o + y'$, Eq. 1 may be rewritten as

$$\frac{1}{g\ b_o\ (y_o + y')}\left[\frac{\partial Q}{\partial t} - \frac{2\ Q}{(y_o + y')}\ \frac{\partial y}{\partial t} + \frac{Q^2}{\left(y_o + y'\right)^2}\ \frac{\partial A}{\partial x}\right]$$

$$+ \frac{\partial y}{\partial x} + \frac{|Q|Q}{C_z^2\ b_o^2\ \left(y_o + y'\right)^2\ \left(R_o + R'\right)} = S_o \qquad (9)$$

Expanding terms dependent on y' and R' in a binomial expansion and neglecting the higher order terms, Eq. 9 after some rearrangement can be written as

$$\frac{1}{g\ A_o}\ \frac{\partial Q}{\partial t} + \frac{|Q|Q}{C_z^2\ b_o^2\ y_o^2\ R_o} + \frac{\partial y}{\partial x} - S_o$$

$$- \frac{1}{g\ A_o}\left[\frac{y'}{y_o}\ \frac{\partial Q}{\partial t} + 2\ \frac{Q}{y_o}\ \frac{\partial y'}{\partial t}\right] - \frac{|Q|Q}{C_z^2\ b_o^2\ y_o^2\ R_o}\left[2\ \frac{y'}{y_o} + \frac{R'}{R_o}\right] + \frac{4\ Q}{g\ A_o\ y_o}\ \frac{y'}{y_o}\ \frac{\partial y'}{\partial t}$$

$$+ \frac{2\ |Q|Q}{C_z^2\ b_o^2\ y_o^2\ R_o} \cdot \frac{y'}{y_o}\ \frac{R'}{R_o} + \frac{Q^2}{g\ A_o^3}\left[b\ \frac{\partial y'}{\partial x} + y_o\ \frac{\partial b}{\partial x}\right] = 0 \qquad (10)$$

Eq. 10 is separated into principal terms independent of y' and R', and first and second order correction terms dependent on y' and R'. It is now convenient to examine each of the correction terms and to determine their influence on the tidal motion. For purposes of illustration, assume that both Q and y' are sinusoidal and there is no steady component of flow. Both Q and y' will then have the same period, so that

$$y' = y_m\ \cos\ (n\ t)$$

$$Q = Q_m\ \cos\ (n\ t - \phi) \qquad (11)$$

where ϕ is the phase angle. Under the conditions of Eq. 11, the first inertia correction term is seen to be

$$- \frac{1}{g\ A_o}\left[\frac{y'}{y_o}\ \frac{\partial Q}{\partial t} + 2\ \frac{Q}{y_o}\ \frac{\partial y'}{\partial t}\right] = -\ y_m\ Q_m\ \frac{n}{y_o}\left[\frac{3}{2}\ \sin\ (2\ n\ t)\ \cos\ \phi\right.$$

$$\left. + \left(\frac{1}{2} - \frac{3}{2}\ \cos\ 2\ n\ t\right)\sin\ \phi\right] \qquad (12)$$

If ϕ is zero, as is the case for a progressive wave, Eq. 12 reduces to $+ \frac{3}{2}\ \frac{Q_m\ n}{g\ A_o}\ \frac{y_m}{y_o}\ \sin\ 2\ n\ t$. Under the same circumstances the principal inertia term is $- \frac{Q_m\ n}{g\ A_o}\ \sin\ n\ t$, so that the correction term is of the order of $\left(3\ y_m\right) /\left(2\ y_o\right)$ times the principal inertia term. This correction in effect introduces a second harmonic distortion which advances the wave crest and retards the wave trough. If the phase angle is greater than zero, a constant is introduced

into the first correction term that is equivalent to a steady slope opposing the bed slope of the channel. Although this type of non-linearity cannot be conveniently introduced into the analog, it is important to note that when the amplitude to depth ratio, y'/y_o, becomes large enough to affect the inertia, the friction increases to the extent that it is usually more important than even the linear part of the inertia.

The second correction term is directly related to the friction. Since R'/R_o is approximately equal to y'/y_o in wide channels, the correction term is on the order of $3\,y'/y_o$ times the principal friction term. Under the assumptions of Eq. 11, the friction term is

$$- \frac{|Q|\,Q}{C_z^2\, b_o^2\, y_o^2\, R_o} \left[2\,\frac{y'}{y_o} + \frac{R'}{R_o} \right] = - \frac{3\, Q_m^2\, y_m}{C_z^2\, b_o^2\, y_o^3\, R_o} \;|\cos n t\,|\cos^2 n t \quad \ldots (13)$$

The expression of Eq. 13 is always negative, and has the effect of piling up water in the direction of wave propagation thus giving rise to the so-called "tidal plane." Physically, this term represents the reduced friction in the direction of wave propagation that results from the large depths normally associated with flood currents. For standing waves, ϕ is 90° and the average friction cor-

TABLE 1.—TYPICAL VALUES OF FROUDE NUMBER IN TIDAL SYSTEMS

Location	F	F^2
Golden Gate	0.050	0.0025
Sea Level Panama Canal	0.122	0.0150
Sacramento River at Ryde	0.059	0.0035

rection is zero as expected. This friction correction, which in some instances is quite significant, can be introduced into the analog with the previously mentioned square-law resistor.

The second correction terms for inertia and friction seen in Eq. 10 are both second order. Consequently, they are small and may be neglected.

The last term of Eq. 10 is dependent on the average depth, y_o, a variation from the average depth, y', and the surface width, b. This last term may also be written in terms of the Froude Number as

$$\frac{Q^2}{g\,A^3} \left[b\,\frac{\partial y'}{\partial x} + y_o\,\frac{\partial b}{\partial x} \right] = F^2 \left[\frac{\partial y'}{\partial x} + \frac{y_o}{b}\,\frac{\partial b}{\partial x} \right] \quad \ldots \ldots \ldots (14)$$

For small values of the Froude Number, the first part of this correction term is completely negligible. The second part of the term depends on the rate of channel widening and must be determined independently for each case. It appears, however, that for small Froude Numbers, the error resulting from neglecting the entire term will be small in relation to such other uncertainties as channel geometry and friction. Some typical values of the Froude Number in tidal systems are listed in Table 1. Unfortunately, this last term which re-

sults in part from convective acceleration cannot be introduced into the analog without using complicated circuits, and an evaluation of its magnitude in relation to the desired results should be made prior to actual construction of the electrical analog.

ANALOG MODEL

To illustrate some of the non-linear aspects of the tidal problem, an electrical analog of the proposed sea-level Panama Canal was constructed (Fig. 1). The canal was assumed to have a length of 220,000 ft and an average depth of 60 ft. Both the Pacific and Atlantic tides were assumed to be sinusoidal and to have a semi-diurnal period of 12 hr.[8] The Pacific tide was further assumed to have a range of 20 ft and to lag the Atlantic tide which had a range of 2 ft by 3 hr. These tides are represented graphically in Fig. 2.

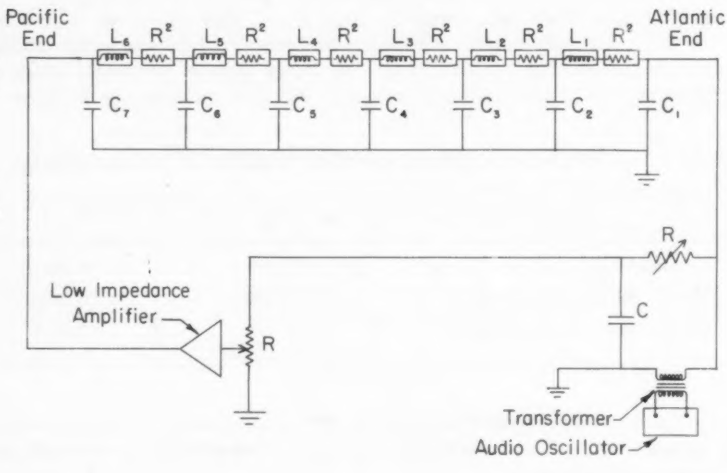

FIG. 1.—ANALOG MODEL

The analog was built into a 21-in.-by-10-in.-by-3-in. metal chassis. The analog chassis and parts were those used in an earlier study by Einstein and Harder of the Panama Canal for the Tidal Hydraulics Committee of the United States Corps of Engineers.[9] In that study the canal was simulated by six equal lumped-parameter electrical elements instead of elements that more closely approximated the individual parts of the hydraulic model sections. However, the wave shapes and amplitudes were nearly indistinguishable from those obtained in the present study. They found that there was no evidence of lumping.

8 "Tidal Currents," by J. S. Meyers and E. A. Schultz, Proceedings, ASCE, Vol. 74, 1948, p. 501.

9 Oral report to the Tidal Hydraulics Committee, U. S. Corps of Engineers, by Hans A. Einstein and James A. Harder, April 28, 1959.

This is as should be expected from previous results[6] because there were 48 electrical elements per wavelength of the principal harmonic.

Each section in the analog consisted of a capacitor, an inductor, and a square law resistor. The average canal properties as determined from the Isthmian Canal Studies and Planning Memoranda 102,[10] and the corresponding values of the electrical elements as computed from Eqs. 6 and 7 are summarized in Table 2. The six sections in the analog were connected together in Pi sections as shown in Fig. 1. The tides were obtained from an audio-oscillator in conjunction with a resistance-capacitance network and a low impedance amplifier. This circuit also seen in Fig. 1 provided the proper phase shift and enabled the tidal amplitudes to be adjusted.

Each of the square law resistors in the analog was set for a Manning "n" of 0.024, the value used in the hydraulic model, by adjusting its characteristics

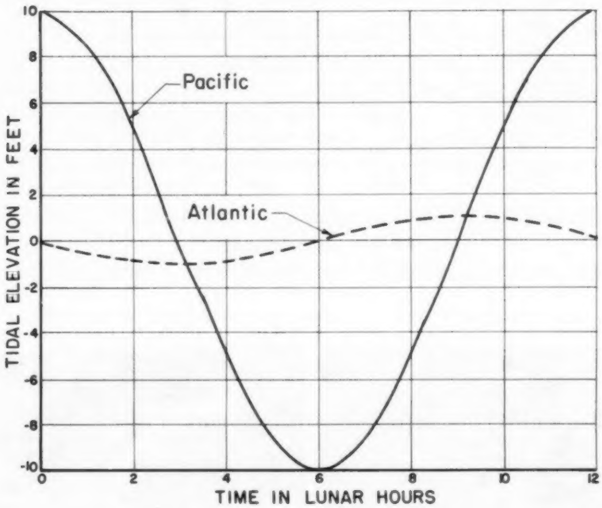

FIG. 2.—BASIC STUDY TIDES

as they were displayed on an oscilloscope screen. As shown in Fig. 3, the vertical deflection corresponds to the discharge, and the horizontal deflection to the energy loss for a depth of 60 ft. The effect of varying the depth is shown in Fig. 3, where the greater energy loss corresponds to a depth of 50 ft and the smaller to a depth of 70 ft. This latter display is obtained by sinusoidally varying the current (corresponding to discharge) at a 60 cycles per sec rate and at the same time varying the voltage (corresponding to depth) at a much higher rate. In this way the energy loss function rapidly varies between its values at 50 ft and 70 ft, giving a band of light on the screen. It is evident from the Manning equation that at zero flow the slope must also be zero. This

10 "Isthmian Canal Studies and Planning Memoranda 102," under Public Law 280-79th Congress, 1st Session, Final Report on Sea-Level Canal Model Tests, 1947.

condition is not accurately obtained with the square-law resistors as seen in Fig. 3. It is believed, however, that the deviation from a zero slope at zero flow is small in relation to the uncertainties in a tidal system as the flow changes direction.

TABLE 2.—SUMMARY OF CHANNEL AND ANALOG CHARACTERISTICS

Section Length, in feet (1)	Average Surface Width, in, feet (2)	Average Cross-Section Area, in square-feet (3)	Average Hydraulic Radius, in feet (4)	frequency = 2500 cfs Inductance, in mili-henrys (5)	frequency = 2500 cfs Capacitance, in micro farads (6)
Atlantic End of Canal					
37,000	860	43,600	50	3.04	0.0117
37,000	770	41,100	51	3.22	0.0222
37,000	740	40,100	51	3.30	0.0206
37,000	780	43,800	52	3.03	0.0202
37,000	690	39,000	52	3.40	0.0195
37,000	975	55,500	52	2.39	0.0227
					0.0133
Pacific End of Canal					

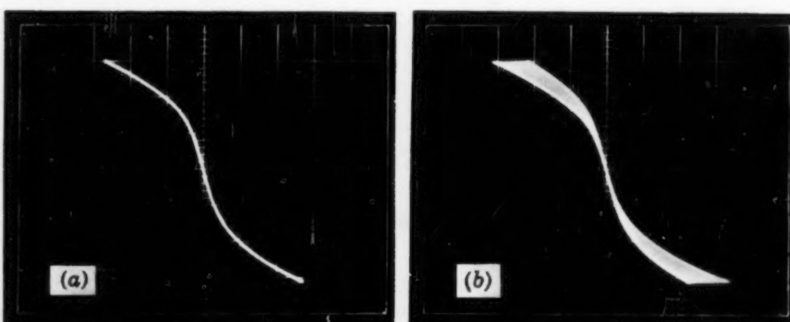

FIG. 3.—SQUARE LAW FRICTION

A probe connected to a differential amplifier and inserted into jacks located between adjacent sections in the analog permitted both voltages and currents to be measured. Both these measurements were taken from the screen of an oscilliscope and converted to corresponding tidal elevations and discharges

by the appropriate scale factors. Mean velocities were then computed from the measured discharge.

ANALOG RESULTS

To verify the data from the electrical analog, tidal elevations and discharges were compared with the results obtained frcm the hydraulic model studies reported in Isthmian Canal Studies and Planning Memoranda 102.[10] It should be noted that this hydraulic model study, the Manning roughnesses of both the model and the proposed prototype were made equal. As similarity principles require that the Manning "n" of the model be reduced by the 1/6 power of the scale ratio, the results of this study are not those that would normally be expected in an actual prototype. Nevertheless, they provided a basis by which the measurements taken from the electrical analog could be compared with those taken from a hydraulic model.

Fig. 4 shows the tidal amplitude as it occurs at the midpoint of the canal from uncontrolled Atlantic and Pacific tides. Although this analog measurement is not compared to hydraulic model data, it does illustrate the flattening of the wave peaks due to square-law friction and the dissymmetry of the wave form that results from a changing water depth. As previously mentioned, the physical explanation for this dissymmetry is the different friction associated with flooding and ebbing currents.

Figs. 5 and 6 show the variation of velocity at the Atlantic and Pacific ends of the canal respectively. A comparison with the data from the hydraulic model study indicates remarkably good agreement between both velocities and wave forms. This is an indication that the non-linear aspects of the flow have been adequately represented in the electrical analog. At the Atlantic end of the canal where the velocities are high, there is a definite flattening of the wave peaks due to square-law friction. The velocities at the Pacific end of the canal are somewhat lower. It should be noted that for the third harmonic of the semi-diurnal tide the canal length is about 0.4 wavelength, close to the half wave resonant length. It appears from the records that the third harmonic phase is shifted, relative to the principal harmonic, so that it flattens the peaks at the Atlantic end and adds to the peaks at the Pacific end.

The dissymmetry of the waveforms brought about by the changing water depth is also seen in Figs. 5 and 6. A dissymmetry between the positive and negative parts of a waveform can be described as the effect of even harmonic components; these cannot be developed by a symmetrical non-linearity such as would be caused by a purely square-law friction element. The ability of the square-law resistor to simulate the effects of changes in the water depth is thus seen to be important in producing the correct wave shape, and to be essential in producing a steady flow through the canal in the absence of a difference in the average water level at the two ends.

Also to be noted from Figs. 5 and 6 is the reliability with which the time of peak velocities can be predicted. If it could be assumed that the hydraulic model results were correct, the maximum deviation between the predicted times of maximum velocity is approximately 4% or less than 30 min in a 12-hr cycle.

Velocity measurements in the hydraulic model were reported for Barro Colorado Island which is near the midpoint of the canal. These results are plotted in Fig. 7 along with the midpoint velocities determined from the analog.

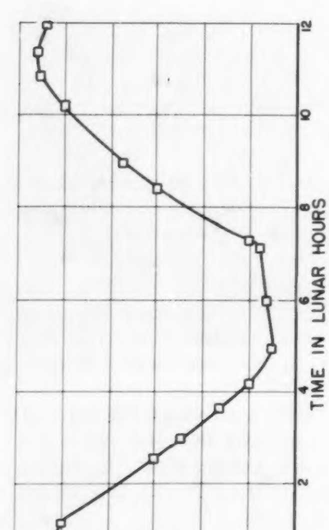

FIG. 4.—TIDAL AMPLITUDE AT CANAL MIDPOINT

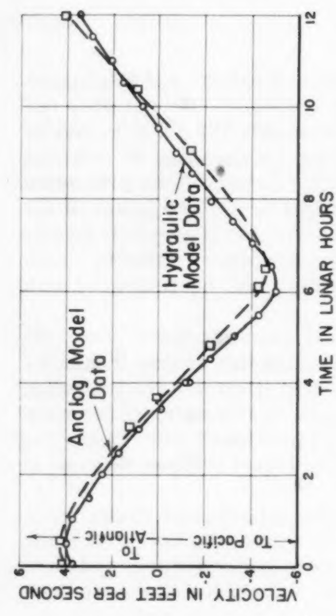

FIG. 6.—MEAN VELOCITY AT PACIFIC END OF CANAL

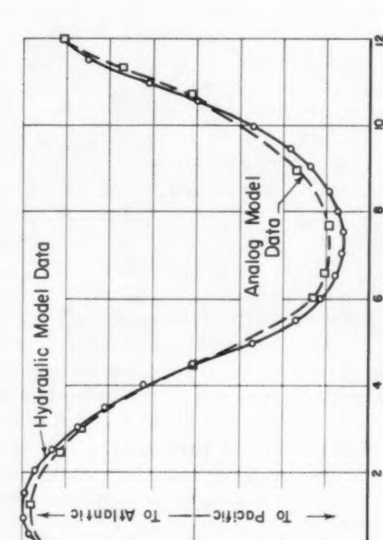

FIG. 5.—MEAN VELOCITY AT ATLANTIC END OF CANAL

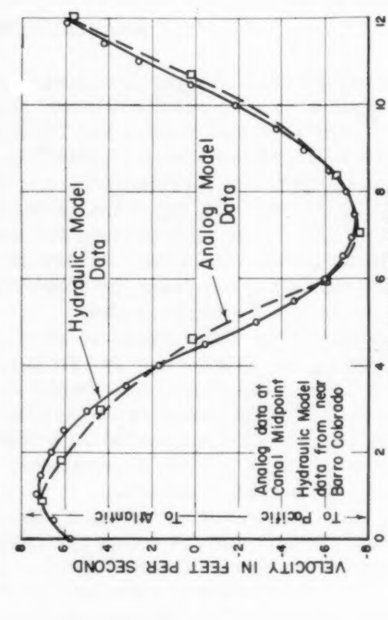

FIG. 7.—MEAN VELOCITY AT CANAL MIDPOINT

Again it is seen that the wave forms have similar shapes and the velocities are of the same order of magnitude.

It is also possible to determine from the analog measurements the phase difference between the discharge and the tide. From Figs. 2, 4, 5, 6 and 7 the discharge is seen to lag the tide by approximately 1 hr or 30°. The uniformity of this phase difference throughout the length of the canal is attributed to the fact that the canal is short in relation the the wavelength.

CONCLUSIONS

Based on the results of this study, it appears that the electrical analog is useful in strictly hydraulic investigations of tidal estuaries. The analog is relatively inexpensive to construct and can be verified in a short time. Accurate descriptions of average velocities, wave forms, and phase differences can be obtained if the non-linear aspects of the tidal problem are included in the analog.

In spite of the case of operation of the electrical analog, a rather complete and thorough knowledge of tidal hydraulics is necessary. In particular, the effects of the various non-linearities on the flow must be understood.

APPENDIX.—NOTATION

The following symbols adopted for use in this paper are listed here for convenience of reference and for use of discussers.

A = Cross-sectional area;

b = surface width;

C = capacitance;

C_o = wave velocity;

C_z = Chezy coefficient;

e = electrical voltage;

i = electrical current;

L = inductance;

Q = flow rate;

R = hydraulic radius;

r = electrical resistance;

t = time in model;

x = horizontal distance along channel;

y = water depth;

ξ = electrical section; and

τ = time in analog.

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LAND AND WATER RESOURCE PLANNING IN TEXAS

By Charles D. Curran,[1] F. ASCE and Forney W. Fleming,[2] M. ASCE

SYNOPSIS

The United States Study Commission-Texas is a new approach by the Federal Government to the coordination of Federal, state, and local interests for area-wide planning for land and water resources. Circumstances leading to its formation are examined. The Commission's objectives, composition, organization, and modes of operation are described and the participating agencies are enumerated. It is concluded that the approach has merit in achieving coordination of multi-level governmental planning.

INTRODUCTION

A Texas historian,[3] has stated that: "Water, the commonest substance of our experience except the air we breathe, has come to be recognized as the most critical single resource of Texas." Because Texas water requirements have enlarged eight-fold in the period from 1930 to 1957, this unequivocal statement will go largely unchallenged by those with the most elementary knowledge of Texas and the water problem with which it is confronted in 1961.

All land and water resource developments in Texas and virtually all of the planning, exclusive of Federal activities, have been accomplished by political

Note.—Discussion open until April 1, 1962. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 87, No. WW 4, November, 1961.

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3 "The Spanish Element in Texas Water Law," Betty Eakle Dobkins, Univ. of Texas Press, Austin, Tex., 1959, p. 1.

subdivisions of the state with limited geographic jurisdiction. Whether under an autocratic government that viewed land and water as the patrimony of the Crown or under one dedicated to the American concepts of liberty; whether planned and built by a city, a county, or a water district, all developments to date have been undertaken by agencies of such limited areal extent that they must be considered local. The activities of the Federal agencies, however, have developed under an entirely different concept, that of primary function.

The greatest developments in the Federal land and water resource activities have occurred since 1940. The increased population and the rapid industrialization and urbanization have been accompanied by increasing demands for navigation facilities, abatement of flood damage and stream pollution, and provision of additional facilities for recreation and for fish and wildlife conservation. These have been the important factors fostering Federal development.

The tremendous proliferation of the Federal activities in land and water resources has resulted in a total of eighteen different agencies now operating in Texas and concerned with some aspects of these resources. Seventeen of these are national in scope and operate in most of the states. One, the International Boundary and Water Commission, United States and Mexico, is unique in that its activities are confined essentially to the Mexican border. Its responsibilities include the development and allocation of international waters. Of the seventeen Federal agencies operating in the study area in 1961, no single one has among its primary purposes the development of all of the uses of land and water that require consideration. The state is divided in two respects: On the one hand, into small geographic segments; and on the other, by different functional considerations. This multiple division is superimposed on an area of great physical diversity.

This diversity of physical characteristics includes differences in rainfall, runoff, topography, and mineral resources. Rainfall, decreasing from east to West, ranges from an average annual of over 55 in. to less than 10 in. Because municipal and industrial water supplies for the future are the core of the Texas water problem, it is essential that the entire area be provided with adequate supplies to support potential additional growth. The southwestern portion of this sector lies in the area of sparse rainfall, and growth may be limited unless importation of water from outside sources is physically feasible and economic.

These considerations led to a study of the Texas water problem by the United States Department of the Interior in 1953.[4] The study, demonstrating considerable vision, concluded by recommending one potential solution through the development and redistribution of the Coastal Plains water resources, a major feature of which was a transbasin water supply canal flowing generally from northeast to southwest.

A valuable subsequence of the study was an estimate of future municipal and industrial water requirements.[5] These estimates were based on an attempt to project future national requirements of products that might be produced in Texas from Texas resources. A portion of this national production was then allocated to Texas on a basin related to its favorable position from the stand-

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4 "Water Supply and the Texas Economy," Senate Document No. 57, 83rd Congress, 1st Session, 1953.

5 "Water for the Future," Bur. of Business Research of the Univ. of Texas, Austin, Tex., Aug., 1956.

point of raw materials and other resources. Water requirements to sustain this production were then estimated. This study, with revisions, has been of continuing value.

The 1953 study was followed by a cooperative study conducted by four agencies: the Bureau of Reclamation, the Corps of Engineers, and the Soil Conservation Service of the Federal establishment, and the Texas Board of Water Engineers.[6]

It constituted, in fact, an inventory of the state's potential water resources, with information on various problems known to exist at that time. Together with the 1953 study, it delineated, qualitatively if not quantitatively, many of the outstanding problems.

PLANNING AGENCIES WITHIN THE STUDY AREA

The continuing development of the nation and the state, accompanied by attendant growth of land and water resource planning, has led to the large number of Federal, state, and local agencies active in the planning field within the study area.

Eighteen Federal agencies, representing five departments and one independent agency, are active in the study area. The major activities of these agencies include collection of basic date, studies and research, construction, and regulation of non-federal activities.

On the state level, nine different agencies are active in land and water resource planning. These nine are composed of six independent boards or commissions, two executive agencies, and an advisory council. Of these nine, only three are exclusively concerned with land and water resource development, and these are independent boards.

Local agencies have, in the past, been responsible for all of the non-federal land and water developments in the study area. These agencies include a large number of municipal governments, over 450 water districts, 18 river authorities operating in eight river basins, and soil conservation districts that include virtually all of the area. In most cases, these local agencies do not, singly, embrace an area of any great size. In fact, only one river authority has jurisdiction over the entire Texas portion of the basin in which it exists. However, most of them are currently fulfilling some function of land and water resource development, and many are planning for the future.

FACTORS LEADING TO ESTABLISHMENT OF COMMISSION

As Federal responsibilities and functions in the field of land and water resource development have been expanded, a basic and continuing problem has been the lack of coordination of all governmental activities in this field. This is a matter not only of lack of coordination within the Federal establishment, but a further lack among the federal, state, and local agencies and the non-governmental interests.

A number of factors have contributed to a lack of unity among the federal agencies. These include statutory limitations, differences of basic function,

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6 "Water Developments and Potentialities of the State of Texas," Senate Document No. 111, 85th Congress, 2nd Session, 1958.

competition for funds and among supports, and the fact that the functions of each agency are the result of unplanned, independent, and uncoordinated growth over a number of years. Federal policy today is an assortment of many separate policies adopted to guide specific programs. The statement of the President's Advisory Committee on Water Resources Policy in 1955 that "the greatest single weakness in the Federal government's activities in the field of water resources development is the lack of cooperation and coordination of the Federal agencies with each other and with the states and local interests" is true today. A number of proposals for remedying this situation have been presented over the years, but nothing completely effective has yet been achieved. Today, an interagency committee in Washington attempts, as have its predecessors since 1939, to coordinate the federal activities. Although its efforts have been helpful, its decisions are not binding on participating agencies, it cannot overcome inconsistencies and incompatibilities in basic federal legislation, and it can do little in Federal-state relationships.

The state of Texas likewise is not yet in position to coordinate all of the planning activities within the State. In 1957 the State Board of Water Engineers was authorized to establish a Texas Water Resources Planning Division. The division was to prepare a report of the water resources of the state and to make recommendation for their maximum development. Additional funds for this assignment were inadequate in total and circumscribed in usefulness by specific salary limitations.

The relatively complete assumption of responsibility for navigation, flood control and related hydroelectric power, and major concern with project type irrigation and affiliated developments by the Federal Government militate against state coordination of planning which includes these functions. Congressional appropriations to discharge these responsibilities are invariably supported by studies of the Federal agency concerned. The state, therefore, is confined to information furnished by the Federal agencies, or to making independent studies that would be duplications of work and expenditure and that would produce results of limited status in the eyes of the Congress.

Despite the lack of a single Federal or state organization capable of coordinating all agencies and interests involved, the necessity for this coordination cannot be overlooked. All of the various agencies combined have a wealth of basic data and previously completed planning that could not be matched short of a joint effort. Furthermore, all of them have interests, objectives, and responsibilities that cannot be ignored in truly comprehensive planning. This necessity is compounded by the large areal scale of planning required; the problem is regional in scope and involves many river basins, each of which is a geographic unit within itself but also culturally tied to the remainder of the state. These considerations in 1958 led to the Congressional authorization of a Commission specifically designed to coordinate all agencies, consider all interests, and study of uses of land and water over the entire study area.

This Commission, which the legislation states "to be known as The United States Study Commission on the Neches, Trinity, Brazos, Colorado, Guadalupe, San Antonio, Nueces, and San Jacinto River Basins and Intervening Areas," has been for convenience called by a short name, United States Study Commission-Texas. The area so defined (Fig. 1) includes the interior watersheds of the state, embraces 62% of its land area, and contains 82% of its 1960 popula-

tion. The excluded portions are drained by interstate or international streams. Two of the interior rivers of the study area actually have their watersheds originating in the neighboring state of New Mexico. Because there is no yield from the out-of-state area, the study area, consists, in essence, of the

FIG. 1.—STUDY AREA

drainage systems in which all runoff originates within the State and over which the State has sovereignty.

DIRECTIVES AND COMPOSITION OF THE COMMISSION

The legislation establishing the United States Study Commission-Texas provides for sixteen Commissioners and, in its membership, brings together sixteen highly qualified representatives from Federal, State, and local interests concerned with water and land development. The Commission is instructed to prepare a comprehensive, integrated plan considering all uses of land and water and all interests involved. It is to cover the eight river basins and the intervening areas of coastal drainage. The law authorizes the

Commission to employ a staff, but also contemplates that it will use the facilities and resources of the existing permanent Federal agencies.

The Commission is instructed to prepare a plan that will provide maximum benefits to the state of Texas and to the nation, to examine the economics of proposed developments, and to offer proposals for the construction of recommended projects, delineating the functions and activities consistent with existing law of the Federal agencies involved.

The Commission is directed to prepare a report setting forth its plan, for submission to the President of the United States. Prior to submission, however, the report must be transmitted for review to the Governor of Texas and to several federal agencies. Comments, views, and recommendations stemming from this review may or may not result in modificaion of the plan; but they must, in any event, be transmitted with the final report to the President.

Ten of the sixteen Commissioners are citizens of the state of Texas not otherwise holding Federal offices. One of these, the chairman, appointed by the President, is required only to be a resident of the study area. Eight are residents of various river basins of the study area and were appointed by the President on nomination by the Governor of Texas. One is a representative of the Texas Board of Water Engineers, nominated and appointed as were the commissioners from the river basins.

Commissioners from within the Executive Branch of the Federal government were appointed by the President from five different departments and one independent agency. The departments represented are: Army; Agriculture; Commerce; Health, Education, and Welfare; and Interior. The independent agency is the Federal Power Commission (FPC). All of the Federal services and bureaus actively engaged in land and water resource development in the study area are included in these six agencies.

These Commissioners have among them a broad knowledge of state and local problems and needs and an intimate knowledge of activities and programs of Federal agencies. They are, however, all appointees of the President and each, as a Commissioner, is responsible to him alone. The legislation states clearly that each Commissioner shall be from a particular area or agency. He is not a representative of or spokesman for an area or agency. This terminology and the underlying distinction of thought have been expressly noted by the Commission. Each Commissioner is expected to be aware that he is not bound by parochical or agency interests and loyalties. His responsibilities are to seek to secure maximum public benefits for the state of Texas and for the nation.

The size of the Commission and the method of selection of its appointees have provided a membership of broad experience and attainment. Any other study commission, similarly selected for another area, might easily produce an equivalent group. The professional and business background encompassed within the Commission may, therefore, be of interest. Eight of the Commissioners are engineers (six of them civil); two are attorneys; two are farmers and ranchers; four are business men, and three are corporation executives. All except one of the nine Commissioners nominated by the Governor have had non-military experience in some level of government.

An early decision of the Commission provided that, in addition to the permanent federal agencies, the services of State and local agencies would be utilized where practicable, and that no studies or surveys that could be performed more efficiently and economically by qualified existing agencies would

be undertaken by the Commission staff. The staff, therefore, is relatively small. Duties of the staff are generally those of supervision, coordination, consultation, and review, but have also included special studies. The review of basic data and determination of methods for filling gaps in these data, objective appraisals of the procedures used by operating agencies, development of acceptable uniform procedures for use in Commission work, the development of criteria, and supervision and review of the completed work to assure conformity with Commission standards are the chief responsibilities of the staff.

This staff total has averaged more than 40 employees and is composed of 25 professional members with the balance made up of subprofessional, clerical, and administrative personnel. The professional group includes specialists from several disciplines—engineering, geology, law, forestry, economics, agriculture, geography, and political science.

The professional staff members have a varied experience and background. Thirteen were resident in Texas when employed, and two others has previous experience in the State. Fifteen have had experience with private enterprise or consulting firms. Experience with eight Federal agencies is included, and several of the staff served with the New York-New England Interagency Basin Committee or the Arkansas-White-Red Interagency Basin Committee. Three have had experience with the Texas Board of Water Engineers. The 25 professional staff members hold among them thirty-four earned academic degrees and thirty certificates of registration as professional engineers, including fifteen issued by the state of Texas. The American Society of Civil Engineers (ASCE) representation on the Commission staff consists of thirteen Fellows and three Members. The staff, although small, thus has a balance of experience in Texas and elsewhere, of Federal and non-Federal backgrounds and of engineering and associated disciplines.

OPERATIONS OF THE COMMISSION

At its initial meeting the Commission established a Planning Coordinating Committee composed of representatives from each of the four principal Federal planning agencies represented on the Commission and the State Board of Water Engineers. This is a committee for technical review, with the Commission's Executive Director designated as chairman. Its designated functions are to advise the Commission on technical matters, to provide coordination during planning, to assure uniformity and consistency in criteria and standards adopted, to make available to the Commission the technical knowledge of the agencies represented, and to review all planning assignments and accomplishments.

The Planning Coordinating Committee reports to the Commission through its Chairman, the Commission's Executive Director, who also heads the Commission staff. The operations of these two units of organization are, in fact, complementary, and the single head for the two is an effective arrangement (Fig. 2).

The staff is divided into three departments—engineering, legal, and administrative. Engineering is further subdivided into three major groups, with water supply and control in one group, water use and economics and water quality in a second, and engineering services and reports in the third (Fig. 3).

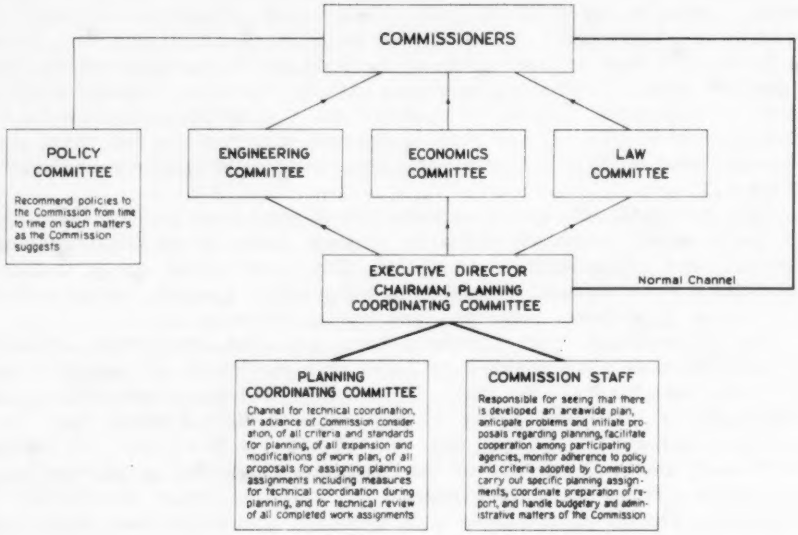

FIG. 2.—ORGANIZATION AND FLOW FOR UNITED STATES STUDY COMMISSION—TEXAS

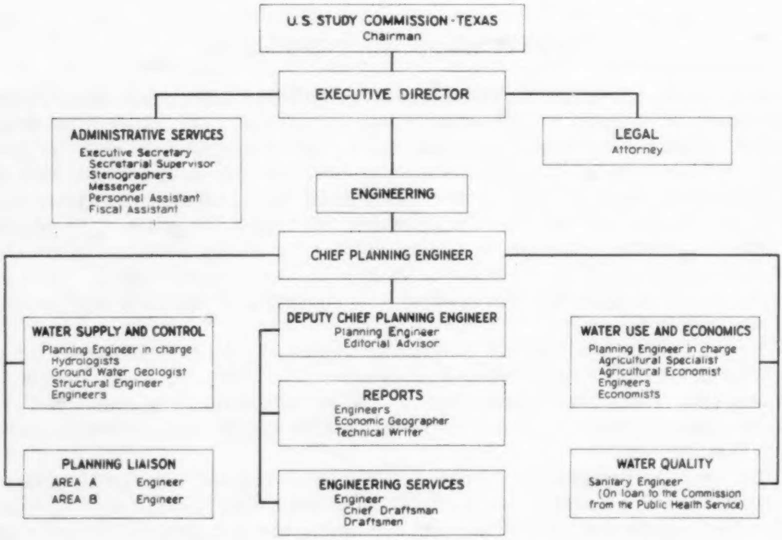

FIG. 3.—STAFF ORGANIZATION

The planning work is being accomplished largely through work assignments to various permanent agencies with particular capacities for accomplishing given tasks. Planning has been divided into two stages, with compilation of basic data comprising the first stage and basin and areal planning the second. In the first task, collaboration groups were tools of major assistance (Fig. 4).

These groups were composed of a staff member, serving as chairman, and a representative from each agency known to be concerned with, or to have pertinent knowledge of, the groups's subject matter. Fourteen of these groups were established for first-stage planning, with the functions of the Sedimentation Group eventually being incorporated into those of the Surface-Water Hydrology Group because of the intimate relationship between the two.

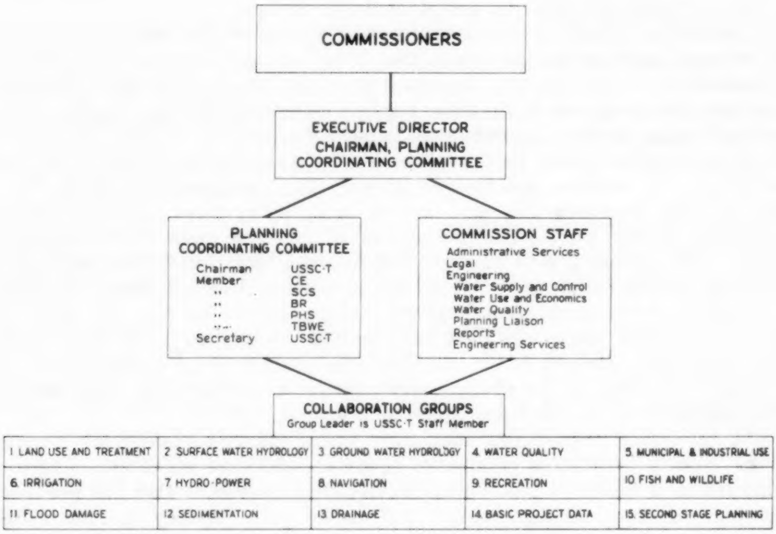

FIG. 4.—COLLABORATION RELATIONSHIP AND FLOW

Represented on these collaboration groups were fifteen Federal agencies, eight State agencies, The University of Texas, the Texas Agricultural and Mechanical College System, and three river authorities.

These collaboration groups generally acted in advisory capacity to the Commission staff. They provided information on the location of basic data in the field of their concern, assessed the value of these data for Commission purposes, and suggested means by which missing data might best be supplied. As a result of their studies, a work assignment was prepared, reviewed by the Planning Coordinating Committee, and submitted to the Commission for approval.

If the work assignment was approved by the Commission, the Executive Director then entered into an agreement with the agency that was to under-

take the particular assignment. The work was then performed by the contracting agency, with monitoring by a staff member to assure conformity to Commission requirements, and a report covering the work done was submitted. This report was then reviewed by the staff and by the collaboration group, and submitted to the Planning Coordinating Committee with the staff and group comments. If the report was deemed satisfactory by the Planning Coordinating Committee and the staff, the Executive Director then submitted it to the Commission, with a summary of the reviews and with his recommendation for action. As a matter of practice none were brought before the Commission until acceptance could be recommended.

Thirty-three of these work assignments were issued for the first-stage planning. These assignments were made to seven federal agencies, The University of Texas, the Texas Board of Water Engineers, the Texas Agricultural and Mechanical College, and the Texas Agricultural Experimental Station, a branch of Texas Agricultural and Mechanical College.

Second-stage planning has been assigned basically to the two largest operating federal agencies in the state, the Corps of Engineers and the Bureau of Reclamation, United States Department of the Interior (USBR). These two agencies have prepared basin plans for four river basins each: the Army Engineers planning for the eastern basins, those of the Neches, Trinity, San Jacinto, and Brazos Rivers; and Reclamation for the western, those of the Colorado, Guadalupe, San Antonio, and Nueces Rivers. The Commission staff is responsible for the areawide plan and is to be assisted by these agencies and their personnel in the work. The collaboration group method has been used for second-stage planning with the Planning Coordinating Committee acting as the partinent collaboration group (Fig. 4). In second-stage planning, opportunity for local interests to express their viewpoints already has been provided. The river authorities' engineers have been invited to a series of informal conferences with the engineers of the planning agencies and of the Commission staff. At these conferences, the preliminary plan for a specific basin has been outlined and the river authorities' comments sought.

The operating precedures used have given the Commission the benefit of advice from a very large number of agencies, and have made available to the Commission a wealth of basic data previously gathered. It also has provided a technical medium through which the major state agency and the Federal agencies have been represented, consulted, and kept fully informed.

Although appropriations for the fiscal year were not available until September, 1960, second-stage planning was actually well underway by then. In the preparation of basinwide plans, both the Corps of Engineers and the Bureau of Reclamation have received assistance from the Soil Conservation Service, the Public Health Service, and the Federal Power Commission in matters involving their competence. Other interested agencies, including the National Park Service, Bureau of Mines and the Fish and Wildlife Service, participate through consultation as opportunities arise. The Army Engineers also have acted as consultants to the Bureau of Reclamation on flood control and navigation features of the western rivers, and Reclamation has played a similar role for irrigation in the eastern basins. Liaison between the various agencies is direct but has been reviewed by staff monitoring.

Because completed planning work is currently (April, 1961) limited, definitive appraisal of the Commission organization and procedures is not practicable at this time. However, the experience already gained warrants certain

conclusions, as well as reservations, on the value of the study commission approach to regional land and water resource planning.

Proposals are before Congress calling for establishment of additional study commissions. In his Budget message sent to Congress on January 19, 1961, President Eisenhower said:

> "To provide for comprehensive, coordinated planning, legislation is being submitted to authorize the President to establish water resources planning commission as needed in the various river basins or regions. These commissions would be composed of Presidentially appointed members from the various Federal agencies and the States. They would prepare and keep current comprehensive integrated river basin plans. This proposed general authority would be an improvement over separate laws such as those which established the two ad hoc river basin study commissions for the Southeastern and Texas areas."

On February 23, 1961, in a message to Congress, President Kennedy said,

> "I urge the Congress to authorize the establishment of planning commissions for all major river basins where adequate coordinated plans are not already in existence. These commissions, on which will be represented the interested agencies at all levels of government, will be charged with the responsibility of preparing comprehensive basic development plans for the next several years."

The immediate problem, therefore, is much broader than that of planning the land and water resources of a regional of the state of Texas. Merits and weaknesses of the study commission as a governmental tool must be examined critically.

Work completed has shown the study commission in Texas to have high value as an organization for the coordination of the water resource planning activities of agencies of all governmental levels. Cooperation of the federal agencies has been achieved since earliest efforts in the basic data collection stage. Acceptance by one agency of data and basic computations of a second has already reduced unnecessary elements of rivalry and has led to economies. The State and local agencies have participated, and their efforts have been coordinated with those of the Federal agencies.

The substantial majority enjoyed by the Commissioners nominated by the Governor assures an opportunity for full consideration of State and local interests. If all levels of government are to assume their share of responsibility for planning and development of land and water resources, assurance that all interests will receive adequate consideration is essential. This desirable feature, however, may be countered by the necessity for a large Commission if on other Commissions there is to be maintained, what might be called, a non-Federal majority. Generally, a small work group is more efficient and less cumbersome. With six Federal agencies represented, a minimum of thirteen Commissioners will be required if this balance is sought.

The Commission provided a means for the assembly of a group of Federal professional careerists freed, for the purposes of the Commission, from restriction by the laws and regulations governing their respective agencies. Incompatibilities and inconsistencies that may be inherent among these laws and regulations need not thwart, or even affect, the Commission's decisions.

The Commission is outside of the regular permanent planning and construction organization of the Federal Government. This gives the Commission greater freedom than it might enjoy if it were within a regular department with its activities there subordinated. If a large number of similar Commissions are established, provision will be needed for executive control and intercommission coordination.

Some commissioners, particularly all those from other Federal agencies, perform their duties as one phase of a full-time employment in land and water resource development, whereas others are engaged in this activity on a part-time basis. These part-time commissioners may bring a broader and a more objective viewpoint to the Commission's deliberations, or they may concentrate their attention on narrow interests of the basin of their residence. They are generally handicapped by lack of the intimate day-to-day familiarity with the problems experienced by those whose Commission work is directly related to their full-time employment.

The Texas Commission is, under present law, to be dissolved on completion of its basic assignment—the preparation of an integrated, comprehensive plan for the development of the land and water resources of the eight-basin area. The maximum value of this plan can be achieved only if it is kept current in the future and is adjusted to changing conditions. The Commission is required to recommend means for keeping the plan current, but that recommendation is yet to be made. Is it realistic to expect any State agency to coordinate the Federal agencies? To what extent is any single regular Federal agency or Federal inter-agency committee capable of coordinating all activities of all levels of government over a regional area? An answer must be forthcoming.

The Commission is a Federal entity. Despite the nomination of nine commissioners by the Governor of Texas, all commissioners are appointees of the President. In the final analysis all Commission expenses are borne by the Federal Government. This factor may have a bearing on the acceptance by the state and local governmental units of any responsibility for the plan and for participation in its accomplishment.

CONCLUSIONS

The scope and complexity of the work, the large number of different agencies involved, the great differences in physical characteristics of the area being studied, and the number of interests requiring consideration combine to provide, within the study area, a suitable testing grounds for the Commission approach. It is clearly a marked improvement over prior devices for coordination. Its existence has permitted initiating procedures that may facilitate a more nearly objective determination of sound development.

Although conclusions might be withheld until the report of the United States Study Commission-Texas is submitted, its organization and methods have demonstrated notable efficacy in the coördination of multi-level governmental activities in regional planning for land and water resource development.

ACKNOWLEDGMENTS

Views and conclusions expressed are those of the writers and do not necessarily represent the views of the United States Study Commission-Texas or of any Commissioner or imply approval or acceptance of the Commission or any commissioner.

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EDDY FORCES ON RIGID CYLINDERS

By A. D. K. Laird[1]

SYNOPSIS

Potential theory is used to approximate forces caused by parallel neighboring cylinders and their eddies on a vertical circular cylinder in waves. The circulation about a cylinder is shown to be an order of magnitude smaller than the circulation of eddies shed by the cylinder.

INTRODUCTION

It has been shown[2] that cylinders in oscillating cross flow experience important alternating drag and transverse forces, in addition to those related to steady state flow.[3] Since there are many possibilities of force variations, because of arbitrary cylinder spacing and complex eddy shedding behavior, the purely experimental method of predicting the forces caused on a piling by wave action is prohibitively laborious. It would, therefore, be desirable to have a method based on theory so that much of the investigation could be aided by high speed computers. It is the purpose of this paper to outline a theoretical meth-

Note.—Discussion open until April 1, 1962. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 87, No. WW 4, November, 1961.

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2 "Water Eddy Forces on Oscillating Cylinders," by A. D. K. Laird, C. A. Johnson, and R. W. Walker, Proceedings, ASCE, Vol. 86, No. HY9, 1960.

3 "Flow about a Pair of Adjacent, Parallel Cylinders, Normal to a Stream: Theoretical Analysis," by L. Landweber, The David W. Taylor Model Basin, U. S. Navy, Report 485, 1942.

od whereby the influences of neighboring cylinders and eddies of the force experienced by a cylinder can be assessed.

Notation.—The symbols used in this paper are defined where they first appear and arranged alphabetically for convenience of reference in the Appendix.

THEORETICAL METHOD

The method consists of predicting the approximate flow field, about one cylinder of a piling, of which the remaining cylinders may be considered neighbors. The forces on the cylinder being considered could then be predicted for any desired arrangments and velocities.

The flow is made up of a steady or oscillating main field with superimposed effects due to the presence of the neighbors and the eddies shed by all the cylinders. If the eddy behavior is known from experimentation, the problem is on a definite basis; otherwise, possible configurations may be specified from the theory with due regard to experience. The simplest approximation might be to assume a von Kármán vortex street from each cylinder. A theoretical and experimental investigation[3] has shown how the von Kármán vortex streets must be modified when the cylinders are abeam in the main flow. Experiments reported[2] in 1960 with cylinders in abeam and tandem arrangements with two or three cylinders subjected to oscillatory flow have shown that further modifications must be made. It was found, in most cases, that eddies tended to be shed in pairs, and that with close spacing, eddy shedding from following cylinders was often suppressed.

The shear layers from cylinders need not be considered in the flow pattern because they have been shown[4,5] to disintegrate close to the cylinder into approximately equal parts of eddies and general undirected motion. A question remains about the extent of a possible "smearing" effect on the contributions of eddies and neighboring cylinders by the general undirected flow in the wake. The results of this paper show, however, that useful predictions can be made without considering this undirected motion.

The strengths of the vorticities of the eddies may be found by the momentum method of Theodor von Kármán, Hon. M. ASCE, but a better method is available[5] which is based on the "notched hodograph method."[6] For a circular cylinder without neighbors, the strength, K, of the vortices based on this method is

$$K = \frac{e\ k^2\ U\ a}{2\ \pi\ S} \quad \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

in which e is the fraction of shear layer vorticity that is converted to eddy vorticity, and has a value of 0.43; k denotes the base pressure parameter, which is the ratio of the velocity on the free streamline at separation to the main stream velocity, and has a value of 1.4; U describes the main stream velocity;

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4 "On the Flow of Air behind an Inclined Flat Plate of Infinite Span," by A. Fage and F. C. Johansen, R and M No. 1104, London, 1927; also Proceedings, Royal Soc. of London, Series A, Vol. 116, No. 773, 1927.

5 "On the Drag and Shedding Frequencies of Two-Dimensional Bluff Bodies," by A. Roshko, NACA T. N. 3169, 1954.

6 "A New Hodograph for Free-Streamline Theory," by A. Roshko, NACA T. N. 3168, 1954.

a is the cylinder radius; and S is Strouhal number based on U and the cylinder diameter,[3] and has a value of 0.21.

For the present examples of 2 in. cylinders performing simple harmonic motion with an amplitude of 9 in. and a period of 6.76 sec, the maximum velocity was 1.37 fps. The first eddies were shed after the cylinders had moved 4 in. from their starting positions at maximum deflection. At the time of shedding, therefore, the cylinders had attained 90% of their maximum velocity; consequently, for simplicity, U = 1.2 fps was taken as a constant for all calculations. The eddy strengths were therefore approximately 0.064 sq ft per sec for the present work.

The circulation about the cylinder, K_C, associated with the warping of the flow field as eddies are shed can be estimated by equating the transverse, or lift force L, of hydrodynamics,

$$L = 2 \pi \rho K_C U \quad \ldots \ldots \ldots (2)$$

to the empirical lift equation

$$L = C_L a \rho U^2 \quad \ldots \ldots \ldots (3)$$

with the result

$$K_C = \frac{C_L U a}{2 \pi} \quad \ldots \ldots \ldots (4)$$

in which ρ is the mass density of the fluid and C_L is the lift coefficient. A typical high measured value of C_L for a cylinder without neighbors was 0.37, from which $K_C = 0.0058$ sq ft per sec.

The preceding values make the ratio $K/K_C = 11$. This important result shows the assumption by some writers that $K_C = K$, in analogy with the shedding of a starting vortex on an airfoil, to be in error by an order of magnitude. Evidence of this discrepancy had been noted as early as 1930 in an investigation[7] in which it was concluded that $K_C < K/4$.

The presence of the neighboring circular cylinders can be accounted for by doublets and numerous infinite series of their reflections in the outlines of the cylinders. Methods have been evolved[8,9,10] whereby these series can be written and manipulated, or approximated. Except for special geometrical spacings of cylinders, these series are impractical to evaluate and must be shortened by choosing only as many terms as are required to give the desired forces on the test cylinder with satisfactory accuracy.

Because the positions of the eddies relative to the cylinders are functions of time, the calculations giving forces as functions of time depend on the locii of the eddies.

As an introduction to the method, two simple cases of practical interest shown in Figs. 1 and 2 will be considered. In Fig. 1, the test cylinder is about

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7 "Eddies behind a Circular Cylinder," by A. Thom, British A. R. C. R and M No. 1373, London, 1930.

8 "Hydrodynamical Images," by L. M. Milne-Thomsen, Proceedings, Cambridge Phil. Soc., Vol. 36, 1940.

9 "Die Reibungslose Stromung in Aussengebiet zweier Kreise," by M. Lagally, Zeitschrift fur angewandte Mathematik und Mechanik, Vol. 9, 1929.

10 "Systeme von Doppelquellen in der ebenen Stromung, insbesondere die Stromungum zwei Kreiszylinder," by W. Muller, Zeitschrift fur angewandte Mathematik und Mechanik, Vol. 9, 1929.

to pass between two eddies of equal strengths, K, shed from a parallel leading neighboring cylinder. The corresponding potential field and the paths of the eddies would be influenced by the circulation on each of the cylinders. The circulation about a cylinder reverses once for each eddy shed from it. For most spacings, the phases of the circulations about the neighbors and about the following cylinder appear to be random and hence unpredictable for any one oscillation. Over a large number of trials, the average eddy path will be that obtained by neglecting the effects of the circulation about all the cylinders. The importance of deviation from this average path in an individual case will depend on the interaction of the circulations of that case, but should be minor for cylinder spacings greater than two or three diameters.

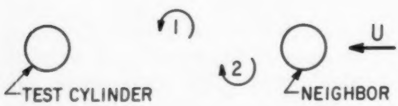

FIG. 1.—CYLINDER AND TWO EDDIES FROM A NEIGHBORING CYLINDER (SCHEMATIC)

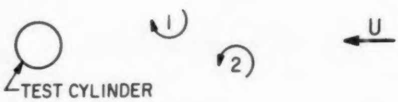

FIG. 2.—CYLINDER AND TWO OF ITS OWN EDDIES (SCHEMATIC)

The simplified equation for the path of eddy 1, obtained by neglecting separation, circulations, eddies shed by the test cylinder, and the presence of the neighboring cylinder, is given by the average stream function,

$$\psi_{1\,avg} = U\, l_1 + \frac{K}{2} \ln \frac{I}{J\ C} = \text{constant} \quad \ldots\ldots\ldots\ldots (5)$$

and for the path of eddy 2,

$$\psi_{2\,avg} = U\, l_2 - \frac{K}{2} \ln \frac{F}{J\ E} = \text{constant} \quad \ldots\ldots\ldots\ldots (6)$$

in which

$$\left.\begin{aligned}
l_1 &= y_1 \left(1 - \frac{a^2}{r_1^2}\right), \quad l_2 = y_2 \left(1 - \frac{a^2}{r_2^2}\right) \\
l_3 &= x_1 \left(1 - \frac{a^2}{r_1^2}\right), \quad l_4 = x_2 \left(1 - \frac{a^2}{r_2^2}\right) \\
l_5 &= y_1 - \frac{a^2\ y_2}{r_2^2}, \quad l_6 = y_2 - \frac{a^2\ y_1}{r_1^2} \\
l_7 &= x_1 - \frac{a^2\ x_2}{r_2^2}, \quad l_8 = x_2 - \frac{a^2\ x_1}{r_1^2}
\end{aligned}\right\} \quad \ldots\ldots\ldots\ldots (7)$$

r is the distance from origin, x denotes the coordinate in direction of U, y is the coordinate normal to x, and

$$I = l_5^2 + l_7^2 \quad \text{(8a)}$$

$$J = \left(x_1 - x_2\right)^2 + \left(y_1 - y_2\right)^2 \quad \text{(8b)}$$

$$C = l_1^2 + l_3^2 \quad \text{(8c)}$$

$$E = l_2^2 + l_4^2 \quad \text{(8d)}$$

and

$$F = l_6^2 + l_8^2 \quad \text{(8e)}$$

The constants, $\psi_{1\ avg} = 0.180$ sq ft per sec and $\psi_{2\ avg} = -0.183$ sq ft per sec, were evaluated for the initial eddy coordinates in Fig. 3(c) using the preceding values of U, K and a. It should be noted that because K contains U to the first power, the eddy paths are independent of the velocity of the main stream.

The velocity u_i and v_i in the x- and y- directions, respectively, of the i^{th} eddy is given by

$$u_i - j\ v_i = - \left\{ \frac{d}{d\,z} \left[w - j\ K \ln\left(z - z_i\right) \right] \right\}_i \quad \text{(9)}$$

in which the last subscript indicates evaluation of the right side at the coordinates of eddy i, z is the complex coordinate, and w is the complex potential. Because the progress of the eddies along their paths as functions of time can be found from the x- components of their velocities, only these need be calculated by the equations

$$u_1 = -U + \frac{U\ a^2 \left(x_1^2 - y_1^2\right)}{G} + \frac{K\left(y_1 - y_2\right)}{J} + \frac{K\ l_1}{C} - \frac{K\ l_5}{I} \quad \text{(10)}$$

and

$$u_2 = -U + \frac{U\ a^2 \left(x_2^2 - y_2^2\right)}{H} + \frac{K\left(y_1 - y_2\right)}{J} - \frac{K\ l_2}{E} + \frac{K\ l_6}{F} \quad \text{(11)}$$

in which

$$G = \left(x_1^2 - y_1^2\right)^2 + 4\ x_1^2\ y_1^2 \quad \text{(12a)}$$

and

$$H = \left(x_2^2 - y_2^2\right)^2 + 4\ x_2^2\ y_2^2 \quad \text{(12b)}$$

The force equations can be calculated for this case, as will be explained, giving the x- and y- force components, F_x and F_y, respectively:

$$F_x = 2 \pi K \rho \left[K_C \left(\frac{x_2}{r_2^2} - \frac{x_1}{r_1^2} \right) + K \left(\frac{l_3}{C} + \frac{l_4}{E} - \frac{l_7}{I} - \frac{l_8}{F} \right) - 2 U a^2 \left(\frac{x_1 y_1}{G} + \frac{x_2 y_2}{H} \right) \right] \quad (13)$$

and

$$F_y = 2 \pi K_C \rho U + 2 \pi K \rho \left[K_C \left(\frac{y_2}{r_2^2} - \frac{y_1}{r_1^2} \right) + K \left(\frac{l_1}{C} + \frac{l_2}{E} - \frac{l_5}{I} - \frac{l_6}{F} \right) + U a^2 \left(\frac{x_1^2 - y_1^2}{G} - \frac{x_2^2 - y_2^2}{H} \right) \right] \quad (14)$$

in which K_C, the circulation around the cylinder, has been included for completeness.

The case of a single cylinder passing between two eddies shed by it during the previous half cycle of its oscillation is illustrated in Fig. 2. The equations corresponding to those for Fig. 1 are found by changing the sign on all the terms containing K; for example,

$$\psi_{1\ avg} = U\ l_1 - \frac{K}{2} \ln \frac{I}{J\ C} = -\ 0.064 \text{ sq ft per sec} \quad (15)$$

and

$$\psi_{2\ avg} = U\ l_2 + \frac{K}{2} \ln \frac{F}{J\ E} = -\ 0.073 \text{ sq ft per sec} \quad (16)$$

in which the constants were evaluated for the initial eddy coordinates in Fig. 3(c). The corresponding force equations in dimensionless form show that the effects of the eddies are independent of the cylinder radius and velocity when S, e, k, and C_L are constant:

$$\frac{F_x}{U^2\ a\ \rho} = \frac{e\ k^2}{S} \left[\frac{C_L}{2 \pi} \left(\frac{X_1}{R_1^2} - \frac{X_2}{R_2^2} \right) + \frac{e\ k^2}{2 \pi S} \left(\frac{L_3}{L_1^2 + L_3^2} + \frac{L_4}{L_2^2 + L_4^2} - \frac{L_7}{L_5^2 + L_7^2} - \frac{L_8}{L_6^2 + L_8^2} \right) - \frac{2\ X_1\ Y_1}{\left(X_1^2 - Y_1^2\right)^2 + 4\ X_1^2\ Y_1^2} + \frac{2\ X_2\ Y_2}{\left(X_2^2 - Y_2^2\right)^2 + 4\ X_2^2\ Y_2^2} \right] \quad (17)$$

and

$$\frac{F_y}{U^2\ a\ \rho} = C_L + \frac{e\ k^2}{S}\left[\frac{C_L}{2\ \pi}\left(\frac{Y_1}{R_1^2} - \frac{Y_2}{R_2^2}\right) + \frac{e\ k^2}{2\ \pi\ S}\left(\frac{L_1}{L_1^2 + L_3^2} + \frac{L_2}{L_2^2 + L_4^2}\right.\right.$$

$$\left. - \frac{L_5}{L_5^2 + L_7^2} - \frac{L_6}{L_6^2 + L_8^2}\right) - \frac{X_1^2 - Y_1^2}{\left(X_1^2 - Y_1^2\right)^2 + 4\ X_1^2\ Y_1^2}$$

$$\left. + \frac{X_2^2 - Y_2^2}{\left(X_2^2 - Y_2^2\right)^2 + 4\ X_2^2\ Y_2^2}\right] \quad \ldots\ldots\ldots\ldots\ldots \text{(18)}$$

in which

$$X_i = \frac{x_i}{a} \quad \ldots\ldots\ldots\ldots\ldots\ldots \text{(19a)}$$

$$Y_i = \frac{y_i}{a} \quad \ldots\ldots\ldots\ldots\ldots\ldots \text{(19b)}$$

$$L_i = \frac{l_i}{a} \quad \ldots\ldots\ldots\ldots\ldots\ldots \text{(19c)}$$

and

$$R_i = \frac{r_i}{a} \quad \ldots\ldots\ldots\ldots\ldots\ldots \text{(19d)}$$

For both cases, the calculations proceeded from the initial coordinates and calculated velocities by assigning a new value of x on the eddy paths and calculating x_2 on the assumption that each eddy traveled at its initial velocity while it was traveling to this second position. The y positions were then found by successive approximations making $\psi_{1\ avg}$ and $\psi_{2\ avg}$ have their given values. After the eddies passed the cylinder, their y-coordinates were held constant at their abeam values. The various constants were evaluated for $K_C = 0$ in a tabular computation resulting in the velocities and forces.

Fig. 3(a) shows the calculated contributions of the eddies to the lift and drag forces for both cases, Fig. 3(b) shows the x-coordinates of the eddies as functions of time, and Fig. 3(c) shows the paths of the eddies past the cylinder. The cylinder in its own wake shows a larger lift, a smaller drag, a simpler force pattern, and a larger eddy velocity than the cylinder following a neighbor.

The starting positions shown correspond to the spacing that would obtain with a simple von Kármán vortex street eddy shedding pattern. To test the effect of the shedding of the eddy pair simultaneously, the forces were calculated with the eddies abeam of the cylinder. With the eddies equally spaced to each side of the cylinder the equations predicted no lift for either direction of circulation. When both eddies were on the same side of the cylinder (at $y = \pm 0.15$ ft and ± 0.32 ft) the eddies from the neighboring cylinder caused a lift force

ratio of 0.375 away from the eddies. Because the force is proportional to the square of the velocity the eddy pair which increased the fluid velocity near the cylinder caused a larger lift than the pair which decreased the velocity. The drag force was zero for all cases because potential theory predicts no drag force when the flow is symmetric about the y-axis. For the actual case an increase in drag would be expected because of the increased velocity past the cylinder when it is between its own eddy pair or to one side of a pair from a neighbor. A decrease would be expected when the velocity was decreased past the cylinder.

The offset eddy position is common where there is a current normal to the orbital velocity of the waves. A cross current of only 1 fps acting during 3 sec could produce the preceding offset positions for 2 ft diameter cylinders.

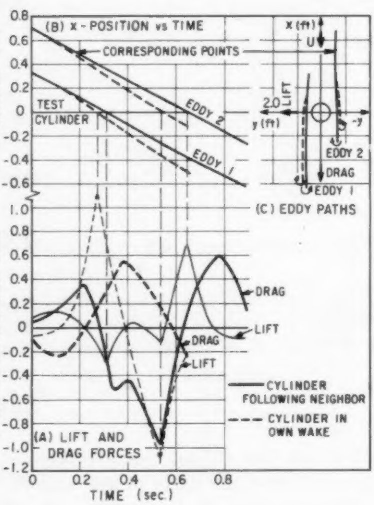

FIG. 3.—CALCULATED FORCES AND POSITIONS FOR CYLINDER IN FIG. 1

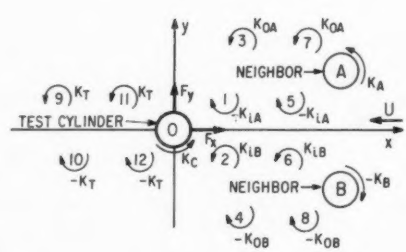

FIG. 4.—TEST CYLINDER WITH TWO NEIGHBORS EACH WITH FOUR SHED EDDIES (CIRCULATION STRENGTHS AS SHOWN)

The magnitudes of the calculated ratios might be altered by about 0.08 for drag forces and about 0.33 for lift forces if the omitted terms containing K_C all had the same sign. The eddies shed by the test cylinder, which were omitted from the calculations, could make only small contributions because they could not be close to the cylinder when the other eddies were. The calculated forces were larger than those measured partly because the vorticity of the eddies was assumed concentrated rather than spread out and diffusing as in nature. Thus the agreement between the measured and calculated forces is satisfactory. This agreement shows the assumptions about the eddy and circulation strengths and the disregard of general wake vorticity did not result in gross errors for the cases considered.

The apparent period of the eddy distrubances for the case with the neighboring cylinder is about half that for the cylinder in its own wake. Also the drag seems to lag the lift force by about a quarter of a period.

As a direct comparison with more complicated data, forces on the test cylinder were calculated from the measured positions and circulation strengths of eddies and cylinders shown schematically in Fig. 4 and to scale in Fig. 5. If the following are neglected:

1. Reflections of neighbors in the test cylinder and in other neighbors;
2. Reflections of the test cylinder in the neighbors;
3. Reflections of eddies in the neighbors; and
4. All higher order reflections;

the complex potential, w, is the sum of the following contributions:

a. Circulation about the cylinders contributes

$$j\ K_C \ln z + j\ K_A \ln\left(z - z_A\right) - j\ K_B \ln\left(z - z_B\right)$$

b. The main stream contributes Uz.

c. Doublets approximating the cylinders contribute

$$U\ a^2\left(\frac{1}{z} + \frac{1}{z - z_A} + \frac{1}{z - z_B}\right)$$

d. The eddies contribute

$$- j\ K_{iA} \ln\left(z - z_1\right) + j\ K_{iB} \ln\left(z - z_2\right) + j\ K_{oA} \ln\left(z - z_3\right)$$

$$- j\ K_{oB} \ln\left(z - z_4\right) - j\ K_{iA} \ln\left(z - z_5\right) + j\ K_{iB} \ln\left(z - z_6\right)$$

$$+ j\ K_{oA} \ln\left(z - z_7\right) - j\ K_{oB} \ln\left(z - z_8\right) + j\ K_T \ln\left(z - z_9\right)$$

$$- j\ K_T \ln\left(z - z_{10}\right) + j\ K_T \ln\left(z - z_{11}\right) - j\ K_T \ln\left(z - z_{12}\right)$$

e. Reflections of eddies in the test cylinder contribute

$$+ j\ K_{iA} \ln\left(z - \frac{a^2}{\bar{z}_1}\right) - j\ K_{iB} \ln\left(z - \frac{a^2}{\bar{z}_2}\right) - j\ K_{oA} \ln\left(z - \frac{a^2}{\bar{z}_3}\right)$$

$$+ j\ K_{oB} \ln\left(z - \frac{a^2}{\bar{z}_4}\right) + j\ K_{iA} \ln\left(z - \frac{a^2}{\bar{z}_5}\right) - j\ K_{iB} \ln\left(z - \frac{a^2}{\bar{z}_6}\right)$$

$$- j\ K_{oA} \ln\left(z - \frac{a^2}{\bar{z}_7}\right) + j\ K_{oB} \ln\left(z - \frac{a^2}{\bar{z}_8}\right) - j\ K_T \ln\left(z - \frac{a^2}{\bar{z}_9}\right)$$

$$+ j\ K_T \ln\left(z - \frac{a^2}{\bar{z}_{10}}\right) - j\ K_T \ln\left(z - \frac{a^2}{\bar{z}_{11}}\right) + j\ K_T \ln\left(z - \frac{a^2}{\bar{z}_{12}}\right)$$

The forces F_x and F_y on the test cylinder can be found by the extended Lagally equation which can be derived by subtracting the sum of the line integrals around each discontinuity outside of the test cylinder from the line integral around a very large circle P, enclosing all the discontinuities. Thus

$$F_x + j\ F_y = \frac{j\ \rho}{2} \left\{ \int_{(P)} \left(\frac{dw}{dz}\right)^2 dz - \sum_{i=1}^{12,A,B} \int_{(i)} \left(\frac{dw}{dz}\right)^2 dz \right\} \quad \ldots\ldots (20)$$

The complex velocity is given by

$$- \frac{dw}{dz} = - \frac{j\ K_C}{z} - \frac{j\ K_A}{z - z_A} + \frac{j\ K_B}{z - z_B} - U + U\ a^2 \left(\frac{1}{z^2} + \frac{1}{\left(z - z_A\right)^2} + \frac{1}{\left(z - z_B\right)^2} \right)$$

$$+ \frac{j\ K_{iA}}{z - z_1} - \frac{j\ K_{iB}}{z - z_2} + \ldots\ldots + \frac{j\ K_T}{z - z_{12}} - \frac{j\ K_{iA}}{z - \dfrac{a^2}{\bar{z}_1}} + \ldots - \frac{j\ K_T}{z - \dfrac{a^2}{\bar{z}_{12}}} \quad \ldots (21)$$

Because the circulation of each eddy and its reflections cancel, the first integral of Eq. 20 gives

$$j\ 2\ \pi\ \rho\ U \left(K_C + K_A - K_B\right) \quad \ldots\ldots\ldots\ldots\ldots (22)$$

The twelve integrals (i = 1 to 12) arising from the eddies in Eq. 20 result in force contributions

$$\begin{aligned} j\ 2\ \pi\ \rho\ \Big[& K_{iA} \left\{ f_1\left(z_1\right) + f_5\left(z_5\right) \right\} - K_{oA} \left\{ f_3\left(z_3\right) + f_7\left(z_7\right) \right\} \\ & - K_{iB} \left\{ f_2\left(z_2\right) + f_6\left(z_6\right) \right\} + K_{oB} \left\{ f_4\left(z_4\right) + f_8\left(z_8\right) \right\} \\ & - K_T \left\{ f_9\left(z_9\right) - f_{10}\left(z_{10}\right) + f_{11}\left(z_{11}\right) - f_{12}\left(z_{12}\right) \right\} \Big] \end{aligned}$$

The circulations about the neighbors directly contribute

$$- j\ 2\ \pi\ \rho \left\{ K_A\ f_A\left(z_A\right) - K_B\ f_B\left(z_B\right) \right\}$$

The presence of the neighbors themselves contribute

$$2\ \pi\ \rho\ U\ a^2 \left\{ f_A'\left(z_A\right) + f_B'\left(z_B\right) \right\}$$

The functions $f_i\left(z_i\right)$, i = 1 to 12, A, B, are the non-singular parts of

$$- \frac{dw}{dz} \text{ at } z = z_i$$

and $f'_A(z_A)$, $f'_B(z_B)$ are

$$\frac{d\{f_A(z)\}}{dz} \text{ at } z = z_A$$

and

$$\frac{d\{f_A(z)\}}{dz} \text{ at } z = z_B,$$

respectively. Each of these 18 functions is made up of about 30 real and 30 imaginary rational algebraic expressions so that the calculation of $F_x + j\,F_y$ from Eq. 20 is probably prohibitive without the aid of a high speed computer, especially since the forces are required as functions of time as the eddies are swept past the test cylinder and the circulation reverses periodically on each of the cylinders.

For the data of Fig. 5, the spacing was such that the strengths of all eddies were equal[3] to K, and if $K_A = K_B = K_C$ with the senses shown, Eq. 20 for the forces becomes

$$F_x + j\,F_y = j\,2\,\pi\,\rho\,U_{MAX}\,K_C + j\,2\,\pi\,\rho\Big\{K\Big[f_1(z_1) - f_2(z_2) - f_3(z_3) + f_4(z_4) + f_5(z_5) - f_6(z_6) - f_7(z_7) + f_8(z_8) - f_9(z_9) + f_{10}(z_{10}) - f_{11}(z_{11}) + f_{12}(z_{12})\Big] - K_C\Big[f_A(z_A) - f_B(z_B)\Big]\Big\} + 2\,\pi\,\rho\,U\,a^2\Big[f'_A(z_A) + f'_B(z_B)\Big] \quad \ldots\ldots\ldots (23)$$

Eq. 23 was used to estimate the forces for a typical set of data[2] as shown, in part, in Table 1. The calculations were done on an IBM 704 computer and checked for one set of values on a desk calculator. The results are compared in Fig. 5 with the measured values. The calculations were carried out for the period after the first two pairs of eddies 1, 2 and 3, 4 were shed until the third pair 5, 6 was nearly abeam of the test cylinder. After 2.16 sec from the start of the cylinders from their positions of maximum displacement, the surface irregularities caused by the wake of the neighbors so blurred the motion pictures that reasonable estimates of the positions of the eddies could not be made. Subsequent eddy positions were found by extrapolating the x- time curves. The measured and calculated lift forces agree satisfactorily after eddy pair 1, 2 passed the cylinder until pair 3, 4 reached the cylinder. During the same time the drop off of the measured contribution of the eddies and neighboring cylinders to the drag forces lags by about 0.13 sec. This discrepancy exceeds the experimental error and is probably a consequence of the simplifications and the fact potential theory predicts no drag for unseparated flow with symmetry about the y- axis. Before 1.9 sec, both the lift and drag display unexplained deviations. After 2.2 sec, the reduced drag measured would not be predicted by the calculations, but should be attributed to the sheltering effect of the wake of the neighboring cylinders.[2]

The central line of the calculated lift force is for no circulation about any of the cylinders. The other two full lines are for K_C positive or negative as indicated. For this configuration of cylinders, the circulations on the neighbor cylinders are as shown[3] in Fig. 4. For considerably wider or narrower

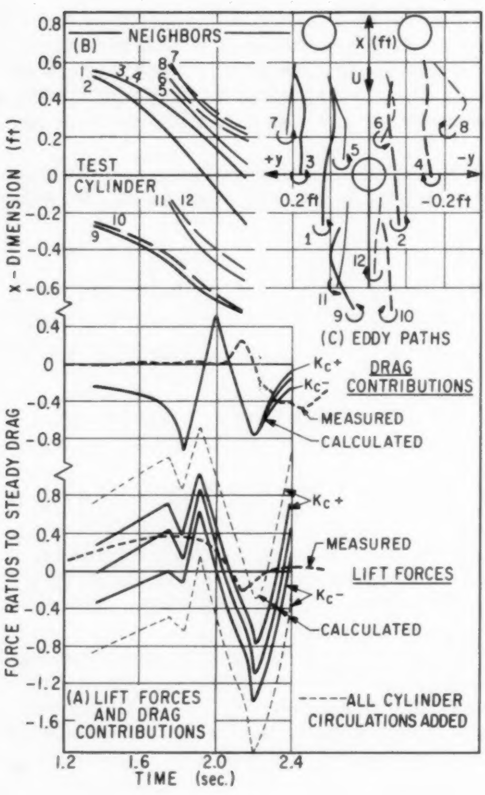

FIG. 5.—MEASURED EDDY AND CYLINDER POSITIONS WITH MEASURED AND CALCULATED FORCES FOR SYSTEM IN FIG. 3

spacings, the circulations on the neighbors are more apt to be in the same sense[3] which, according to Eq. 22, could cause a $\pm$ lift on the test cylinder of an additional 49% of the steady state drag. It is probable that these maximum predicted lift forces were not encountered because of the statistically small

number of runs in which lifts were measured.[2] More drag forces were measured, however, so the predicted maximum values were more closely realized.

Because neither the eddies shed from a nearly round cylinder, nor the forces on the cylinder are sensitive to the exact shape of the cylinder,[10] it is sufficient to show that the complex potential, previously used, has nearly cir-

TABLE 1.—TYPICAL MEASURED EDDY POSITIONS AND CALCULATED FORCES OF FIG. 4

Time[a]		1.34		1.75		1.91		2.16	
Coordinates[b]		x	y	x	y	x	y	x	y
Eddy[c]	1	0.54	0.22	0.23	0.17	0.04	0.23	-0.27	0.23
	2	0.52	-0.13	0.25	-0.10	0.03	-0.14	-0.26	-0.17
	3	0.58	0.40	0.39	0.32	0.22	0.36	0	0.35
	4	0.61	-0.30	0.39	-0.32	0.25	-0.30	0	-0.34
	5			0.47	0.19	0.28	0.12	0.08	0.13
	6			0.52	-0.10	0.32	-0.14	0.19	-0.07
	7			0.61	0.39	0.38	0.41	0.22	0.43
	8			0.59	-0.35	0.38	-0.52	0.25	-0.44
	9	-0.25	0.13	-0.45	0.19	-0.62	0.13	-0.72	0.07
	10	-0.26	-0.08	-0.41	-C.21	-0.57	-0.21	-0.74	-0.23
	11			-0.14	0.10	-0.40	0.13	-0.57	0.17
	12			-0.13	-0.07	-0.32	-0.05	-0.50	-0.04
Drag Ratio[d]									
	$K_c > 0$	-0.231		-0.453		-0.279		-0.620	
	$K_c = 0$	-0.234		-0.485		-0.272		-0.631	
	$K_c < 0$	-0.237		-0.519		-0.265		-0.645	
Lift Ratio[d]									
	$K_c > 0$	0.226		0.717		1.049		-0.424	
	$K_c = 0$	-0.060		0.374		0.859		-0.675	
	$K_c < 0$	-0.346		0.034		0.672		-0.923	

Neighboring cylinders at x = 0.75 ft, y = ± 0.25 ft.

[a] Time (in seconds) measured from start of oscillation at maximum deflection.

[b] Eddy coordinates (in feet) measured from test cylinder with x positive upstream.

[c] Numbers refer to eddies shown in Figs. 4 and 5.

[d] Force ratios are calculated forces divided by drag force for steady state with C_D = 1.3 and U = 1.2 fps. Machine results rounded to 3 places.

cular stagnation streamlines around O, A and B which represent the outlines of the cylinders.

If the small effect of the eddies on the shapes of the stagnation streamlines are neglected, the stream function divided by Ua becomes

$$1 - \frac{X}{X^2 + Y^2} - \frac{Y - Y_A}{(X - X_A)^2 + (Y - Y_A)^2} - \frac{Y - Y_B}{(X - X_B)^2 + (Y - Y_B)^2}$$

in which X = x/a and Y = y/a. At stagnation points the velocity is zero. From

the x- component of velocity,

$$1 = \frac{X^2 - Y^2}{(X^2 - Y^2)^2} + \frac{(X - X_A)^2 - (Y - Y_A)^2}{\left\{(X - X_A)^2 + (Y - Y_A)^2\right\}^2}$$

$$+ \frac{(X - X_B)^2 - (Y - Y_B)^2}{\left\{(X - X_B)^2 + (Y - Y_B)^2\right\}^2} \quad \ldots\ldots\ldots\ldots \quad (24)$$

From the y- component of velocity,

$$O = \frac{X\ Y}{(X^2 + Y^2)^2} + \frac{(X - X_A)(Y - Y_A)}{\left\{(X - X_A)^2 + (Y - Y_A)^2\right\}^2} + \frac{(X - X_B)(Y - Y_B)}{\left\{(X - X_B)^2 + (Y - Y_B)^2\right\}^2} \ldots \quad (25)$$

For the cylinder spacings of Fig. 5, Eq. 25 gives $Y = 0$ for the location of the stagnation points near 0. The distance of the leading stagnation point upstream from 0 can be found by setting $X = 1 + \epsilon$ in Eq. 24 and the trailing stagnation point is downstream from 0, a distance $X = -1 + \delta$. The terms ϵ and δ are very much less than unity. The algebra is simplified by neglecting ϵ^2 and δ^2 compared to ϵ and δ, respectively, giving $\epsilon = 0.0090$ and $\delta = -0.0067$. Similar calculations place the stagnation points on the neighboring cylinders at $Y = \pm 3.000$, $X = 9.991$ and 8.007. These distortions of the cylinder outlines were considered negligible. The neglected eddies and reflections should not make significantly larger distortions.

The calculated forces based on a constant stream velocity, U, are for the set of cylinders in a stream, but the results apply also to cylinders in waves for which the orbital excursions of the water particles are large compared to the cylinder diameters. The data[2] did not contain the phase effect that could occur with waves, whereby the eddy could be shed from a neighbor near the time of maximum velocity past the neighbor and be carried across the test cylinder near the time when the velocity was a maximum past it. It is to be expected that occasionally all the eddy, neighbor, and phase effects will be additive.

It should be emphasized that the analysis in this paper is for rigidly supported cylinders which do not themselves deflect or vibrate because of the applied forces. If the cylinders are not rigid, coupling is possible and resonant effects may cause very much larger forces.[3,11]

CONCLUSIONS

For the conditions of this investigation the following conclusions may be drawn:

1. The flow pattern about rigid cylinders and their eddies can be represented by potential flow equations that are sufficiently simple to make possible

11 "Water Forces on Accelerated Cylinders," by A. D. K. Laird, C. A. Johnson, and R. W. Walker, Transactions, ASCE, Vol. 125, Part I, 1960.

analytical force predictions that are sufficient to account for corresponding measured forces in streams and wave action.

2. The possibility of neighboring cylinders and eddies increasing drag forces by 100%, and causing lift forces equal to steady state drag forces has been predicted for rigid cylinders.

3. The eddies shed from a single rigid cylinder can cause large additional forces when swept past the cylinder by subsequent wave action.

4. The paths of the eddies are independent of the steady stream velocity which generates them.

5. The effect of general vorticity in the wake apparently may be neglected without impairing the usefulness of the calculations.

6. The circulation about a rigid cylinder is of the order of one eleventh of the circulation of an eddy shed by it.

ACKNOWLEDGMENTS

The author is indebted to the University of California Computer Center, Berkeley, Calif., for the use of their facilities and to Mr. H. R. Gillette for advice on the computer.

APPENDIX.—NOTATION

The symbols used in the paper are listed here for convenience of reference and for the aid of discussers:

A,B	= locations of neighboring cylinders;
a	= cylinder radius;
C,E,F,G,H,I,J	= algebraic expressions defined in text;
C_D,C_L	= drag and lift coefficients, respectively;
e	= fraction of shear layer vorticity converted to eddy vorticity;
F_x,F_y	= force components on text cylinder in x- and y- directions, respectively;
$f'_A(z_A)$	$= \frac{d}{dz}\left\{f_A(z)\right\}$ at $z = z_A$;
$f'_B(z_B)$	$= \frac{d}{dz}\left\{f_B(z)\right\}$ at $z = z_B$;
$f_i(z_i)$	= non-singular part of $-\frac{dw}{dz}$ at $z = z_i$;
i	= general index or subscript for eddies;
j	= square root of minus unity;
K	= strength of eddy;
K_A,K_B	= strength of circulation about neighboring cylinders at A and B, respectively;

K_C	= strenght of circulation about the test cylinder;
K_T	= strength of eddy shed from test cylinder;
K_{iA}, K_{iB}	= strength of eddy shed from the inboard sides of the neighboring cylinders at A and B, respectively;
K_{oA}, K_{oB}	= strength of eddy shed from the outboard sides of the neighboring cylinders at A and B, respectively;
k	= base pressure parameter = ratio of the velocity on the free streamline at separation to the main stream velocity;
L	= lift per unit length = $\pm$ y- component of force on test cylinder per unit length of cylinder;
L_i	= dimensionless algebraic expression $(l_i = aL_i)$;
l_i	= algebraic expression (i replaced by numerical subscript for identification);
ln	= natural logarithm;
O	= origin of coordinates and location of test cylinder;
P	= large circle enclosing all discontinuities in a field;
R_i	= dimensionless distance from origin $(r_i = a\ R_i)$;
r	= distance from the origin of coordinates;
S	= Strouhal number based on the main stream velocity and cylinder diameter;
U	= main stream velocity taken as 90% of U_{MAX};
U_{MAX}	= maximum velocity of the oscillating cylinders;
u,v	= local velocities in the x- and y- directions, respectively;
w	= complex potential;
X,Y	= dimensionless coordinates (x = aX, y = aY);
X_i, Y_i	= dimensionless coordinates of eddy locations $(x_i = a\ X_i,\ y_i = a\ Y_i)$;
x	= coordinate in the direction of the main stream velocity;
y	= coordinate normal to x and the cylinder axis;
z	= complex coordinate = x + jy;
$\overline{z}$	= complex conjugate of z $(\overline{z} = x - jy)$;
δ, ϵ	= dimensionless increments in X- and Y- directions, respectively;
π	= circumference of a circle divided by its diameter;
ρ	= mass density of the fluid; and
$\psi_{i\ avg}$	= stream function defining an average path of all eddies that could be shed at the same given point during many oscillations.

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Proceedings of the American Society of Civil Engineers

NAVY'S NEW CARRIER BERTHING FACILITIES AT SAN DIEGO

By D. R. Forrest,[1] F. ASCE, and John B. Stetson,[2] M. ASCE

SYNOPSIS

The planning and construction of the Navy's new berthing facilities for Forrestal-class aircraft carriers at San Diego are described. These facilities include installation of a retractable fender system, camels and cranes, modification of utilities at an existing wharf, and dredging of a turning basin and of 6.7 miles of deep-draft channel in San Diego Bay.

INTRODUCTION

By December, 1961, the Navy will have completed a $4,250,000 dredging and berth modification project in San Diego Bay that will permit use of the harbor by the Navy's largest aircraft carriers. San Diego, headquarters of the Eleventh Naval District, has one of the best natural harbors of the Pacific Coast. However, a minimum depth of 35 ft presently limits use of the harbor to Essex-class carriers and vessels of lesser draft. The new project depth will be 42 ft, mean lower low water. The project consists of three parts: (a) dredging of an 800-ft wide 2.5-mile entrance channel, (b) dredging of a 600-ft

Note.—Discussion open until April 1, 1962. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 87, No. WW 4, November, 1961.

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wide 4.2-mile bay channel and turning basin, and (c) modification of berthing facilities at the Naval Air Station, North Island.

HISTORICAL DEVELOPMENT OF SAN DIEGO BAY

The history of San Diego Bay must begin with Juan Rodriguez Cabrillo, Portugese explorer in the employ of Spain, who landed at Ballast Point on September 28, 1542. He appraised the harbor as "closed and very good." A brief visit was again paid to San Diego 6 months later by Ferrelo, chief pilot for Cabrillo, following the latter's death on San Miguel Island. The next visitor was Sebastian Viscaino. With 200 men and 3 ships, Viscaino entered the bay in 1602. In accordance with his consistent practice of changing practically all names of places designated by his predecessor, Viscaino changed the original name of San Miguel given by Cabrillo to San Diego. Viscaino recorded that he found a minimum depth of 21 ft over the outer bar at the bay entrance and a quiet anchorage inside Ballast Point. There does not appear to have been further exploration of the bay until the Spanish military base and the Mission San Diego were established in 1769. The first United States Naval vessel to enter the harbor was the U.S.S. Betsy in 1800.

Until the channel was modified by dredging, ships entering San Diego Bay passed Ballast Point and found two channels separated by a middle ground. The main channel, lying to the west of the middle ground, was of ample depth but difficult to navigate. East of the middle ground the channel was crooked and changeable. Inside the harbor the channel was about one-third mile wide and not less than 30 ft deep all of the way to the town of San Diego.

The first improvement of the harbor of importance was commenced in 1875 by the Corps of Engineers and was completed the following year. The work consisted of diverting the San Diego River from the harbor by cutting a channel and building a levee, causing the river to empty into False (now Mission) Bay. In the annual report of the Chief of Engineers, United States Army, of February 1, 1888, he notes ". . . The general condition of the embankment is good, but in need of repair. It has been injured by the burrowing of animals and the wash of rain storms, and a portion of the rock facing has also been thrown down. Should the appropriation of $1,000 asked for in my last annual report be made, it will be expended in making these necessary repairs. . ." The same report contains comments on a survey and estimated cost of obtaining a channel 250 feet wide and 24 feet deep across the outer bar, such investigation being a proviso of the River and Harbor Act of August 5, 1886. The report further states ". . . To control a portion of the ebb flow beyond the heads and give it a direction so as to produce a greater working effect upon the bar, it is proposed to construct a jetty on Zuninga (Zuniga) Shoals, about 7,500 feet in length, which will carry it out to the 15-foot curve. . ." Construction of the jetty was authorized by Congress in 1890 and work was commenced by the Corps of Engineers in 1893. Work continued intermittently under several contracts until completion in 1904. No further work was done on the jetty until 1941, when 500 ft of the inshore end was increased in elevation.

The first Federal dredging project was begun by the Corps of Engineers in 1891 and consisted of widening and deepening the entrance channel across the outer bar to a minimum depth of 24 ft. Subsequent dredging by the Corps of Engineers, completed in 1934, increased the entrance channel width to 800 ft and deepened it to 40 ft.

Over the years, many dredging projects by the City of San Diego, the Navy and the Corps of Engineers provided 35 ft of water from the entrance channel to the northern portion of the bay, 30 ft in the central portion and lesser depths elsewhere. In the process, tidelands were reclaimed, including the northwestern portion of North Island; the Spanish Bight between North Island and Coronado; the eastern portion of Coronado; the present site of the Naval Amphibious Base; much of the Naval Station; Shelter Island; and Dutch Flat on which is now

FIG. 1.—AERIAL VIEW OF NORTH SAN DIEGO BAY SHOWING THE NAVAL AIR STATION NORTH ISLAND IN THE FOREGROUND WITH THE EXISTING SHELTER ISLAND DEVELOPMENT AT THE LEFT AND THE LOCATION OF THE NEW HARBOR ISLAND ADJACENT TO LINDBERGH FIELD

situated portions of Lindbergh Field, the Marine Corps Recruit Depot and the Naval Training Center. Fig. 1 provides a general view of the North Bay area as it appeared in 1960.

PROJECT REQUIREMENT AND SITE SELECTION

In 1954, the U.S.S. Forrestal (CVA-59) was launched at Newport News Shipbuilding and Dry Dock Company. This vessel was the first of a new class of

aircraft carriers having a length of 1038 ft, a draft of 37 ft and a standard displacement of 54,600 tons. While solving the Navy's problems concerning the requirements of the newer carrier aircraft, these vessels presented many unique problems to the Navy's port and harbor engineers.

The first vessels of this class assigned to the Pacific Fleet were homeported at Naval Air Station, Alameda, in San Francisco Bay where adequate depth of water was available at the existing carrier berths. As plans were developed to utilize more of these large carriers in the Pacific, it became apparent that berthing facilities for these vessels would be required at the Naval Air Station, North Island, in San Diego Bay for reasons of dispersal and to insure maximum utilization of existing major naval air stations on the west coast. Because harbor entrance channel depths and turning basin depths in San Diego Bay were totally inadequate for the Forrestal-class carriers, engineering studies were undertaken in 1956 to determine the most feasible and economical means of providing the required berthing facility in this area.

Selection of a site at North Island for this facility was influenced by various factors including economy of initial construction, future costs and overall effectiveness of the facility to fulfill its assigned mission. Based on these general factors, a site just inside the harbor across from Ballast Point seemed to offer the most feasible solution. This location offered ready access to the open sea, a suitable bay area for the required turning basin, and the necessary proximity to existing support facilities. Being located away from the existing carrier berths, some dispersal within the bay would be realized. The site offered an additional advantage from the construction standpoint in that dredged material from the turning basin could be profitably disposed of east of Zuniga Point jetty in an area that could later be utilized to extend the Naval Air Station's Runway 18-36.

The project as planned at this stage encompassed a 42-ft entrance channel 800 ft in width, a 2500-ft diameter turning basin and a 600-ft cellular wharf structure with two mooring platforms. The general layout of this preliminary plan is shown in Fig. 2. The total cost for the project as planned was estimated at approximately $7,500,000.

Although an optimum solution appeared to have been reached, the possibility of utilizing existing berthing structures in the bay area was investigated in an attempt to achieve overall economies in initial construction costs and to eliminate logistic problems that would result from the somewhat remote location at Zuniga Point. Although other structures in the area were eliminated for various reasons, the quay wall, which was then serving as a berth for two Essex-class carriers, seemed to offer some interesting possibilities. In spite of the additional channel dredging that would be required, preliminary estimates indicated that savings in the amount of $2,500,000 could be realized through using this existing structure in lieu of constructing a new berthing facility. Although minor modifications to the wharf would suffice, it was recognized that new berthing spaces for the Essex-class carriers would have to be found and that the channel would conflict with other Navy and commercial traffic, existing fleet moorings, and seaplane operations in the North Bay area. Logistic problems that would have been encountered at the Zuniga Point site would be practically eliminated, however, by utilizing the quay wall. This advantage, combined with the lower initial cost, the ability to berth two CVA-59 carriers rather than only one as planned at the previous site, and the realization that if the channel were not dredged, the present well-developed support area at North

Island would become of less value as the older carriers were eventually replaced, turned the scales in favor of the new location. An additional factor that undoubtedly aided in enlisting popular support for the project was the completion of the City of San Diego's Tenth Avenue marine terminal. The Navy's proposed deep water channel to the quay wall would naturally be of major benefit to long range City plans for their facility.

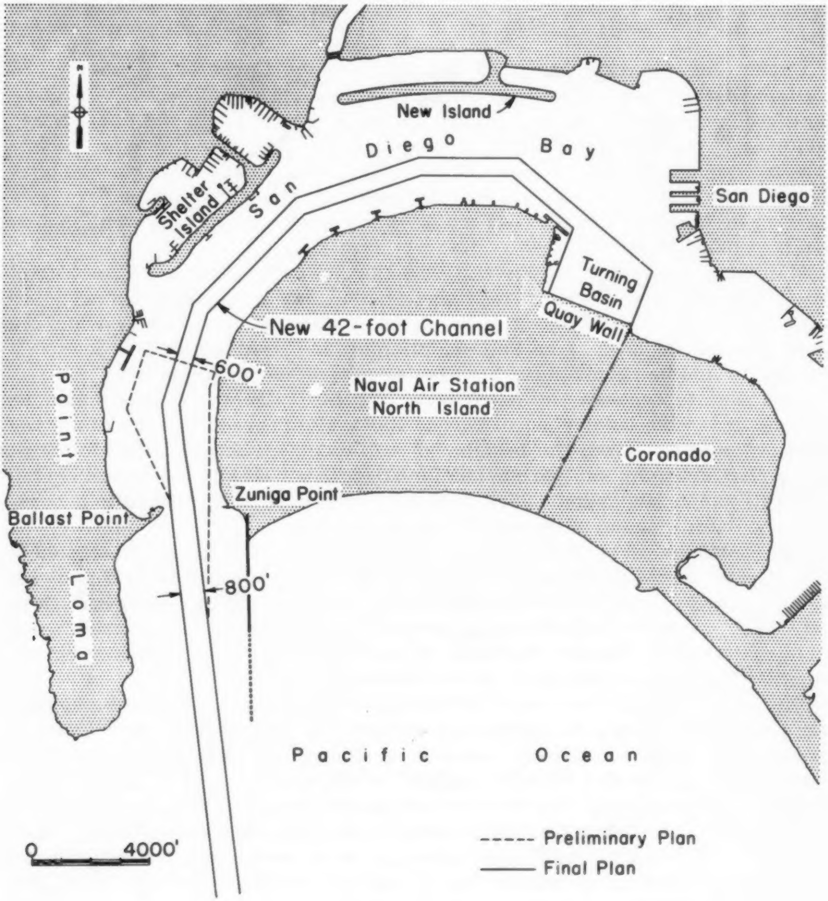

FIG. 2.—GENERAL PROJECT PLAN SHOWING PRELIMINARY AND FINAL DREDGING AREAS

By November, 1959, the situation had become critical. Several large carriers had been assigned to the Pacific Fleet and an additional Forrestal-class carrier was scheduled for deployment in 1961. This additional vessel had to be homeported in San Diego for strategic reasons. After careful con-

sideration of the advantages and disadvantages of each site, the decision was made in December, 1959, to utilize the existing quay wall, modified as required. In addition to these modifications, the project included the originally planned 800-ft entrance channel seaward of Ballast Point with a 600-ft channel and a 2500-ft diameter turning basin within the bay as indicated on Fig. 2. The estimated cost for this scheme was $4,500,000.

PROJECT PLANNING

During the planning stage of the project, the Corps of Engineers, Los Angeles District, was asked to estimate the frequency of maintenance dredging likely to be required in maintaining the new project depth and to study and comment on the probable effects the proposed dredging would have on navigation and on the stability of adjacent shorelines. The Corps of Engineers concluded that neither navigation nor adjacent shorelines would be adversely affected by the work and that bay maintenance dredging probably would not be required more frequently than at 10-yr intervals.

Maintenance dredging of the entrance channel in the vicinity of the ancient bar (to minus 40 ft) was last accomplished in 1948. Minor shoaling has occurred since that time. The Corps suggested that a depth of 40 ft may be about the limit for the channel to be self-maintaining and dredging at 5-yr intervals probably would be required for the greater project depth in this area.

Exposure to occasionally heavy seas and the extreme pumping distances encountered in the outer channel beyond Ballast Point precluded use of a pipeline dredge for that area. The dredging job was therefore divided into two parts: The harbor dredging was earmarked for accomplishment by competitively bid contract, and arrangements were made with the Corps of Engineers to accomplish the outer channel work with a seagoing hopper dredge.

The northern part of San Diego Bay is well developed, as illustrated in Fig. 1, and disposal sites for dredged material within economical pumping distances are becoming increasingly difficult to find. The cities of San Diego and Coronado were invited to suggest sites within their jurisdiction which, if filled, would be of benefit. The sites suggested, in addition to areas on or adjacent to Federally owned land, were evaluated.

After the comparative study of various combinations, it became evident that creation of an island using the major portion of the dredged material would provide the most economical solution. Realizing the excellent commercial possibilities that a new harbor island offered, the City of San Diego readily accepted this proposal. The Harbor Department is now proceeding with plans for a development similar to the existing Shelter Island including a small craft marina, roads, utilities and lease space for such ventures as restaurants and motels. This site, as shown in Figs. 1 and 2, offered the obvious advantages of centralized location with respect to the dredging areas and a sufficient capacity to receive nearly the entire dredged quantity. The material placed in this area is allowed to take its natural underwater slope of 6 or 8 to 1 and is diked at 3 to 1 above elevation plus 3. The City of San Diego will ultimately shape the fill to suit its desired purposes and will provide the necessary revetments.

Ballast Point Cove near the harbor entrance is the future location of a nuclear submarine facility which will consist of an enclosed landfill area and a 550-ft pier. The high quality dredged material available in the adjacent new

channel would be ideal for the proposed land fill and, therefore, it was decided to stockpile 120,000 cu yd for that purpose. An additional 180,000 cu yd were designated for disposal along the beach in the Ballast Point Cove, outside of the area to be developed for the nuclear submarines where it would be available as fill for any future development. The remaining 100,000 cu yd to be dredged from adjacent portions of the channel were destined to be placed on a narrow beach and shoal north of the submarine pier site. Neither of these latter areas required bulkheading; however, diking was specified in conjunction with the stockpiling operation to keep the material in close proximity to its area of future use.

Hydrographic surveys were made with the use of a Raytheon fathometer. Seventy-three borings were made at 500-ft intervals, to a depth of 46 ft and samples of material encountered were retained for bidder inspection. Three important limitations were placed on bidders. First, the new channel will be approximately in the middle of the fairway, and the dredging contractor would be required to use submerged discharge lines in order to minimize obstruction to navigation. Second, the time schedule imposed would require a production rate after mobilization of over 500,000 cu yd per month. Third, the work near Ballast Point would have to be accomplished at the beginning of the job in order to provide fill material in adequate time for use in the nuclear submarine pier contract.

BAY CHANNEL DREDGING

Bids for the bay dredging portion of the project were opened on December 22, 1960. The low of four unit-price bids received was $0.664 for dredging 3,460,000 cu yd. Award was made to the low bidder and the completion date set at September 27, 1961.

Dredge pay quantities are being computed from hydrographic surveys made by fathometer immediately before and after dredging, at approximately monthly intervals. The specifications permit payment at the bid price for overdredging to a maximum of two feet. Quantities are calculated on the basis of surveyed ranges spaced at 100-ft intervals. After-dredging surveys include ranges at 50-ft intervals to assist in determining whether project depth is obtained. Survey control consists of a series of monumented base lines established along the perimeter of North Island. Distance along a range while sounding is determined by tag-line.

The dredging of a deep water channel constituted a major problem in the Shetler Island area where Navy anchorages had been established. Because vessels anchored in this area would swing into the channel with the tide, it was necessary to lay bow and stern moorings along Shelter Island to accommodate nine ships. In addition, the tracing and removal of communication and other submarine cables proved to be an interesting but time consuming occupation to the engineers associated with this project. A degaussing range across the channel near Ballast Point consisting of 24 tubes with interior coils and connecting cables was typical of the obstructions encounterd.

ENTRANCE CHANNEL DREDGING

As indicated previously, dredging of the entrance channel seaward of Ballast Point could be best accomplished by seagoing hopper dredge. The only such

dredges on the Pacific Coast are four Government-owned vessels operated by the Portland District of the Corps of Engineers. These dredges generally work in the Pacific Northwest harbors during the summer and in San Francisco Bay, or elsewhere as needed, during the winter. Because dredging schedules are determined many months in advance, early negotiation and assignment of a dredge was essential. The hopper dredge Chester A. Harding, shown in Fig. 3, was assigned temporarily to the Los Angeles District, Corps of Engineers, for this purpose and began work in the San Diego Bay entrance channel on March 21, 1961.

FIG. 3.—U. S. ARMY CORPS OF ENGINEERS DREDGE CHESTER A. HARDING BERTHED IN SAN DIEGO PRIOR TO COMMENCING DREDGING OPERATIONS

The Harding, built in 1939, is of 2,720 cu yd capacity, has two 2,120 hp diesel propulsion engines driving twin variable-pitch screws, and is equipped with two drag heads and two 650 hp diesel-driven pumps. She is 308 ft long and has a 56-ft beam. Her master is Carl Heil, a veteran of 32 yr with the Corps of Engineers.

By mid-May, approximately 1,000,000 cu yd of material had been removed to provide a minimum depth of 42 ft throughout the entire 800-ft wide channel.

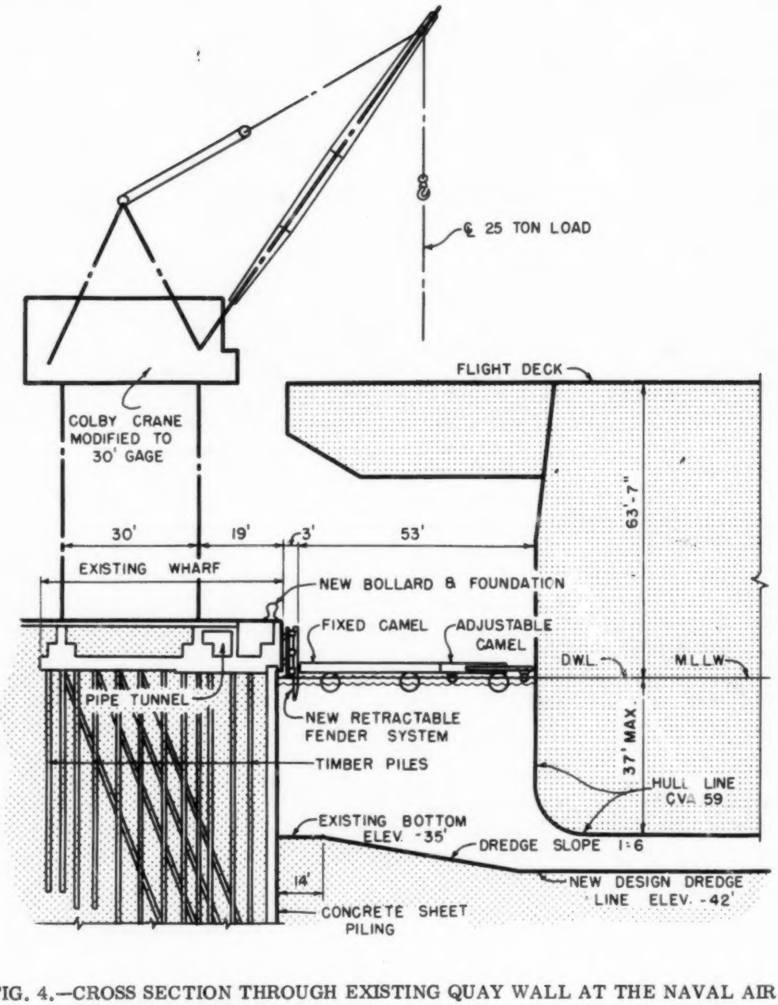

FIG. 4.—CROSS SECTION THROUGH EXISTING QUAY WALL AT THE NAVAL AIR STATION NORTH ISLAND

The material was dumped at sea approximately 3 miles offshore in about 250 ft of water. In spite of delays caused by the presence of a large number of cobbles, abandoned cable, and other debris that clogged the drag heads and pumps, daily production averaged 21,300 cu yd. Fuel, provisions and other supplies and services were furnished through Naval supply facilities. The United States Coast Guard cooperated in adjusting the location of channel buoys so that they would be clear of the dredging area. Survey control for the dredge and periodic condition surveys were made by the Los Angeles District, Corps of Engineers.

WHARF MODIFICATIONS

As engineering progressed on the dredging portion of this project, a separate set of contract plans and specifications were being prepared by the consulting engineers, for modifications to the quay wall where the large aircraft carriers are to be berthed. The existing marginal wharf is a relieving platform structure on timber piles with a concrete sheet pile bulkhead at the seaward face. The structure was investigated and found to be adequate for all anticipated loads. The original design had been based on ultimate dredging to -40 ft. Because the carriers would be held 56 ft from the marginal wharf by special camels, the anticipated dredged slope of 1 on 6 from -44 ft (allowing two feet overdredging) would intersect the existing -35 ft bottom approximately 14 ft from the face of the bulkhead and therefore no strengthening of the wharf was required. Fig. 4 is a general cross section of the wharf showing these relationships. Structurally, the only major changes in the wharf consist of providing adequate bases for the 100-ton bollards that are required for mooring the CVA-59 class carriers. Based on experience at the New York Naval Shipyard and other Navy installations, a gravity type fender system was specified to replace the existing fender piles. Fig. 5, a closeup of the completed fender system, shows the slotted bracket that is characteristic of this type of fendering. In the utility field, a new salt water pumping station, consisting of two 1750 gpm pumps at 180 psi, was specified. This station was connected to existing salt water outlets by a new 12 in. distribution line. The wharf fresh water system was found to be adequate. However, backflow preventers were installed on all existing fresh water outlets to protect the station's water distribution system in case of accidental cross connection with a non-potable system aboard ship. The electric power distribution system on the wharf was modified as required to meet the heavier demands of the large aircraft carriers, the new pumping station and the two electric drive portal cranes, that were designated for installation on the wharf. No changes in the existing crane rails were required; however, because the cranes are driven by 125 hp electric motors, a cable trough was specified by which the cranes would receive power through a cable and reel arrangement.

A contract for these modifications was awarded on November 30, 1960, in the amount of $776,000. The work was completed in July, 1961. Fig. 6, showing an aircraft carrier berthed at the quay wall, provides an indication of the scope of the project.

Vessels at the marginal wharf had been served by 25-ton portal cranes as shown at the right in Fig. 7. The 64-ft height of the CVA-59 flight deck above the water line and the anticipated lifts which would be required in serving these heavy carriers dictated procurement of cranes with greater height of

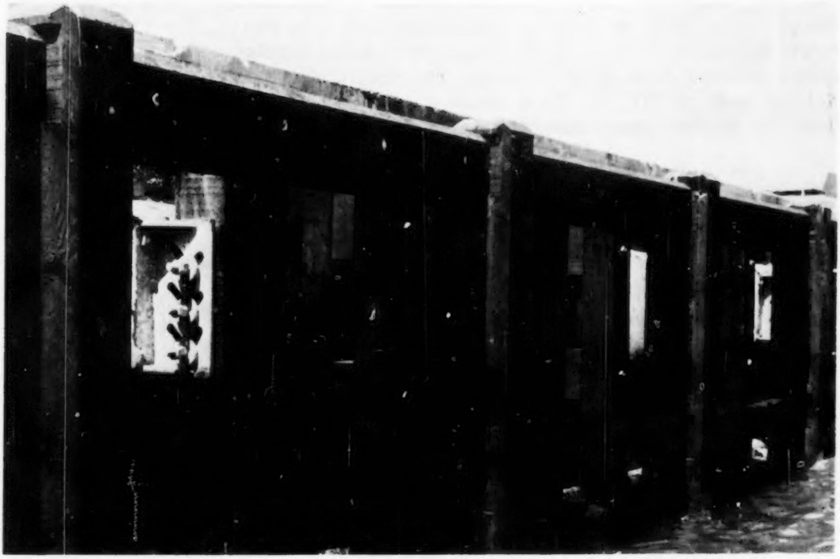

FIG. 5.—CLOSE-UP OF COMPLETED GRAVITY-TYPE FENDER SYSTEM AS INSTALLED ON THE QUAY WALL, NAVAL AIR STATION NORTH ISLAND

FIG. 6.—AIRCRAFT CARRIER USS LEXINGTON MOORED AT COMPLETED MARGINAL WHARF, NAVAL AIR STATION NORTH ISLAND

lift and increased capacity. Two surplus Colby 50-ton, model 240, portal cranes were obtained from the Naval Shipyard, Long Beach. The cranes, which were designed for 24-ft gauge rails, were disassembled and modified for use on the existing rails at 30-ft gauge. The modifications were accomplished for a total cost of $97,100. This phase of the work was completed in May, 1961, and the cranes were immediately placed in service as indicated by Fig. 7.

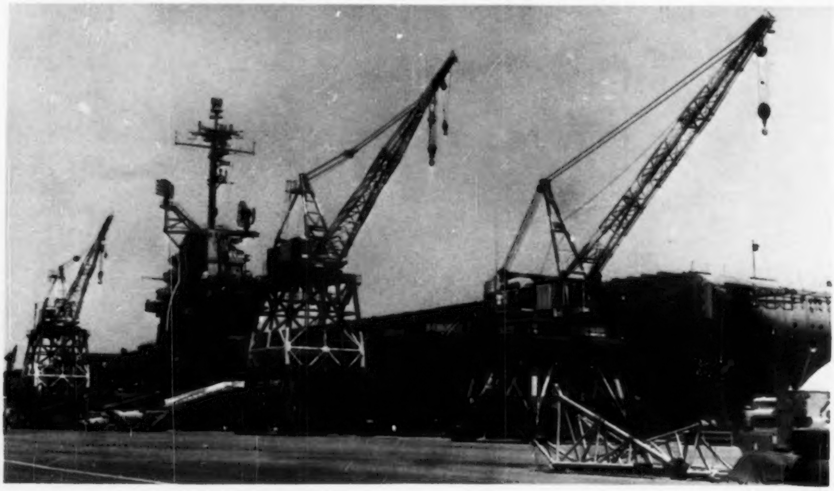

FIG. 7.—GENERAL VIEW OF WHARF AREA SHOWING 25-TON CRANE IN FOREGROUND AND THE TWO MODIFIED 50-TON COLBY CRANES SHORTLY AFTER COMPLETION OF THEIR CONVERSION

FIG. 8.—DREDGE SEATTLE WORKING IN THE TURNING BASIN AREA, NORTH SAN DIEGO BAY

As the dredging and wharf modification work progressed, one additional problem remained to be solved. This concerned the marking of the deep water channel. Originally, it was considered that shore ranges would provide the most feasible solution. However, because of the several course changes dic-

tated by the geography of the bay, and because of occasional fog and the possibility of poor visibility, plus potential confusion with city lights in the background at night, lighted channel buoys were adopted. These aids to navigation are being obtained and installed by the United States Coast Guard.

CONCLUSIONS

At the present time (1961), Navy design engineers can see tangible results from their continuing efforts since 1956. Modifications to the quay are complete, and two 50-ton portal cranes painted in the typical orange-and-white

FIG. 9.—HARBOR ISLAND DEVELOPMENT

checkerboard now adorn the marginal wharf. The Corps of Engineers hopper dredge Harding has achieved project depth in the outer channel except for a few shoals that could not be dredged by the Harding because of numerous large cobbles. The two most important shoals presently are being removed by means of a clam-shell dredge under separate contract. Within the harbor, the hydraulic dredge Seattle, shown in Fig. 8, with the dredge Dallas as a booster, is rapidly creating the new 70-acre island, Fig. 9, and has completed the channel from Ballast Point to the turning basin as well as the major part

of the turning basin. Moorings have been relocated and the stockpile of dredged material has been placed at the site of the nuclear submarine pier. The current cost estimate for the entire project is $4,250,000. Through this project, by December 1961, San Diego Bay will be capable of accommodating the world's largest aircraft carriers.

ACKNOWLEDGMENTS

Planning, design, and construction for this project have been accomplished under the direction of the Director, Southwest Division, Bureau of Yards and Docks, San Diego, Calif. D. R. Forrest and Harold Spires were the Navy Project Design Engineers.

Contract plans and specifications for the harbor dredging and wharf modification projects were prepared by the firm of Moffatt & Nichol, Engineers, of Long Beach, Calif., whose Project Engineers were James W. Dunham and S. V. Bailey. Frank's Dredging Company, Long Beach, Calif., was responsible for the bay channel dredging. The M. H. Golden Construction Company, San Diego, Calif., handled the wharf modifications. The Westmont Engineering Company, Bell, Calif. modified the cranes following designs prepared by Colby Steel and Manufacturing, Inc.

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DISCUSSION

PNEUMATIC BARRIER AGAINST SALT WATER INTRUSION[a]

Closure by Ian Larsen

IAN LARSEN.[7]—The writer was interested to hear of the experiments conducted by T. M. Dick, in which a value of α = 0.3 was determined in the relation

$$q_s = \alpha \, h_s \sqrt[3]{v\, g} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (31)$$

The difference between this value and that obtained by G. I. Taylor was not surprising, however. Taylor's theory was based on the assumption that "the bubbles are so small that they rise much more slowly through the surrounding water than the currents they produce" so "they may be expected to act simply by reducing the mean density of the mixture of water and bubbles.[8] Clearly, in practice, the bubbles would be of such a size that this theory could not hold exactly, and the dimensionless factor α, would depend on the excess pressure of air and the diameter and distance between holes in the pipe. By varying these factors enough it was possible in fact, to obtain values of α ranging from 0.2 to 0.8.[9] The value of α = 0.3 obtained by Dick, thus, seemed to be only applicable to certain combinations of excess pressure and hole diameters and spacings.

In fact, the value of α in Eq. 31 was not in itself measure of the efficiency of the pneumatic installation. From this point of view, indeed, the theory of Taylor had been largely replaced in practice by the experimental results of V. D. Burgh,[9] which served better for calculating actual efficiencies.

Concerning sedimentation, the writer agreed that in rivers subject to heavy sedimentation, bars of sediment could be formed around the pipe. For an estuary in regime, however, these could scarcely build up to any extent. Moreover, for many rivers and estuaries this problem did not even arise.

The use of the pneumatic barrier in tidal estuaries posed special problems, but sufficient progress had now been made to show its practicability. A complete description could well form the subject of another paper, and the subject could only be outlined briefly in this reply.

A "penetration length" of the salt water in an estuary was defined as the maximum distance traversed by the salt water-fresh water boundary. The

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a September 1960, by Ian Larsen (Proc. Paper 2600).

7 Research Engr., Coastal Engrg. Lab., Tech. Univ. of Denmark, Copenhagen, Denmark.

8 "The Action of a Surface Current Used as a Breakwater," by Sir Geoffrey Taylor, Proceedings, Royal Soc. of London, Series A, Vol. 231, 1955, p. 466.

9 "Proeven met Samengeperste lucht te Lelystad en Gorinchem," by V. D. Burgh, Rijkswaterstaat, 's-Gravenhage, 1960.

penetration length may then be shown to tend towards a minimum as the vertical variation in salinity tends everywhere towards zero. It is this effect which is achieved by increasing the vertical mixing with the rising layers of air bubbles.

The study of these processes could be pursued in a model so long, of course, as the scale factors could be sufficiently determined. The writer could state that these factors were now sufficiently well-defined to allow the use of model studies.

DESIGN OF INLETS FOR TEXAS COASTAL FISHERIES[a]

Closure by H. P. Carothers

H. P. CAROTHERS.[33]—The discussion by F. Gerritsen is of special interest with respect to "stability shear stress." This parameter is fully described elsewhere.[34]

An average stability shear stress of 0.093 psf is indicated for stable inlets in that publication.[34] This parameter is based on maximum flow at spring tide. In the discussion, the use of formula 3, Design Velocity, is seriously questioned as giving too low a shear stress. But this design velocity is applied to median (50%) tidal differential, not to maximum flow at spring tide conditions.

Along the central Texas coast, median tidal differential is 0.42 ft, with diurnal (tropic) tide range of 1.5 ft. Also hourly analyses of typical tropic tides show a maximum tidal differential of 1.5 ft. It is interesting to note that preliminary application of the formulation in subject paper indicated a 10-ft depth for the proposed Corpus Christi Fish Pass, with a "stability shear stress" of 0.082 psf. Subsequent detailed application through the full range of tidal differentials of formula 6, Sediment Transport, including dynamic balance with the Gulf bar, now indicates a final design depth of 12 ft, with a "stability shear stress" of 0.095 psf. This compares favorably with the suggested average value of 0.093 psf.

It is considered that these comparative analyses using the parameter "stability shear stress," suggested by the discussion, provide a significant and most encouraging substantiation of the formulation presented in subject paper.

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[a] September 1960, by H. P. Carothers and H. C. Innis (Proc. Paper 2603).

[33] Hd., Special Proj. Dept., Lockwood, Andrews & Newnam, Inc., Cons. Engrs., Houston, Tex.

[34] "Stability of Coastal Inlets," by P. Bruun and F. Gerritsen, Proceedings, ASCE, Vol. 84, No. WW 3, May, 1958.

RUSSIAN RIVER CHANNEL WORKS[a]

Closure by Israel H. Steinberg

ISRAEL H. STEINBERG.[10]—Von Seggern has presented a detailed and vivid description of his personal experience with investigating, planning and designing various types of channel works. A major part of his observations were on the Salinas River where different types of installations had been constructed over a period of years with some degree of indicated success. These works included wood and steel jacks, tetrahedons and also the flexible fence developed by H. E. Rowe and Walter Gerow as mentioned in the discussion by R. Robinson Rowe, F. ASCE.

The pioneering work by Von Seggern during his association with the San Francisco District Corps of Engineers, formed the basis for the selection of the alternative types of installations for the initial test reach on the Russian River. Supplementing the work conducted by Von Seggern during 1939 to 1945, was an evaluation of the bank protection work on both the Salinas and Pajaro Rivers in 1953 and 1954 as a part of a civil works investigation program of the Corps of Engineers. In addition, a model study, of limited scope, was conducted by the University of California under a contract with the San Francisco District to investigate the fundamentals of the action of river training structures. Although the results were not fully conclusive, the indications were that channel training works require wings, as well as main line jacks, to be more fully effective (Fig. 7 for illustration). The wings intercept some of the debris and tend to slow down the velocity of the water which enters the area landward of the main jack lines (Fig. 9). In contrast to this, the channel training works on the Salinas mainly have consisted of a single row of main line jacks.

The installation shown in Fig. 7 is similar to that on the Rio Grande in the vicinity of Albuquerque which the writer had the opportunity to inspect in May, 1955.

The remedial measures referred to in the paper were completed in the fall of 1960. The 1960-61 flood season, however, produced only moderate flood flows, it being the least severe of occurrence since the construction of the initial reach in 1956. There was no real opportunity, therefore, to fully evaluate the effectiveness of these remedial measures.

One significant condition did develop which warrants some mention. Partial flanking, with some attendant bank erosion, had been noted at river mile 54.5 where tree pendants anchored at the top and bottom by longitudinal cables had been installed. Remedial measures consisted of salvaging of the anchor cables, trimming and backfilling the eroded portion of bank and the placing

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[a] November 1960, by Israel H. Steinberg (Proc. Paper 2647).

[10] Asst. Chf., Planning and Reports Branch, U. S. Army Engr. Dist, Corps of Engrs., San Francisco, Calif.

of new pendants along the damaged section. The new pendants were anchored to the salvaged cables which were realized along the bank after backfilling was completed. As a further measure, the portion of the new work near the bottom of the bank was covered with wire mesh. The top edge of this line of mesh was attached to the lower longitudinal cable and the bottom edge held in place by a series of 8-in. cubes of concrete, on 10-ft centers, lying in the channel bottom.

Notwithstanding the occurrence of only moderate flood flows, the reconstructed works still proved vulnerable to the action of the river and a section again failed. Because similar types of works which were constructed at river miles 53.7 and 55.8 are relatively successful, the conclusion can be reached that the velocities and general direction of flow against the bank at this location are severe enough to cause undercutting of the toe and eventual sloughing of the bank. In future planning, some additional care will need to be exercised in the selection of the sites at which bank stabilization by means of tree pendants is to be considered.

Errata.—The following revisions should be made in the text of the writer's paper: page 31, last sentence under "Check Dams" should read: "Remedial measures being planned involve the placing of riprap around the ends of the abutments. The riprap will extend a distance of about 25 ft upstream and downstream from the check dam."

SACRAMENTO RIVER DEEP WATER CHANNEL: FUNCTIONAL PLANNING[a]

Closure by Amalio Gomez

AMALIO GOMEZ,[4] F. ASCE.—Bower's discussion raised two points: The first relates to the unit benefit for deep draft traffic. The second relates to the method of computing navigation benefits.

Bower assumes that the computed unit benefits for deep draft cargo represent the difference between the rates charged to the user or shipper and the cost of transporting the same commodity by water. The assumption is not justified from the data given in the paper. The correct procedure is to compute the cost to the nation of transporting a given commodity by water and by the cheapest alternative means. The difference in cost represents the unit benefit to the nation. The cost of transporting a given commodity by land may or may not be accurately reflected in freight rates. Even if such were the case, the question of whether or not freight rates should be discounted by the overhead component depends largely on whether the conditions in the particular locality are those of a nearly static, mature economy, or whether they represent a growing economy in need of additional transportation facilities. Therefore, a complete study is needed for each commodity in each case. These complex and lengthy studies are outside the scope of the paper. The principle that the writer wanted to illustrate was that navigation benefits represent the difference in cost to the nation in transporting a commodity by land and by water.

The second point raised by Bower ralates to the computation of average annual benefits from a given growing tonnage and a fixed unit benefit. The procedure suggested by Bower is, of course, correct, and is standard practice with Federal agencies. However, the procedure was not fully explained by the writer and it is understandable that Bower misunderstood it. The procedure actually used is as follows: (a) A year by year tonnage projection was made for each commodity; (b) such tonnage for each commodity was multiplied by the appropriate unit savings to get total benefits for each year; (c) the present worth of such benefits was obtained and converted into an equivalent 50-yr annunity; and (d) the value of this annuity was divided by the expected tonnage at the midpoint of the 50-yr economic life of the project. The value of annuity previously, computed was $2,060,000. The tonnage at the midpoint was 857,000 tons, and the weighted equivalent unit benefit was 2,060,000/857,000 = $2.40. The figure of $3.17 given in the paper is the sum of $2.40 and $0.77 and might be considered to represent the weighted equivalent unit cost of land transportation as applied to the midpoint tonnage. The development and significance of this figure were not properly explained in the paper.

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[a] November 1960, by Amalio Gomez (Proc. Paper 2649).

[4] Chf., Planning and Reports Branch, Engrg., Div., U. S. Army Engr. Dist., Sacramento, Calif.

The writer is grateful to Bower for having raised two points which apparently were not adequately covered in the paper.

Errata.—The following typographical errors appear in the original paper: page 57, line 13, change "fo" to "of;"page 58, line 21, change "slough" to "sloughs;" page 58, line 22, change "tow" to "toe;" and page 63, Condition E, line 3, change "600,000" to "500,000."

LATEST DREDGING PRACTICE[a]

By Arthur L. Collins and Closure by Ole P. Erickson

ARTHUR L. COLLINS,[9] M. ASCE.—It is inconceivable that the large pipe line dredges have a defect which robs the industry of an estimated 25% of the potential output. The dredge Western Chief is an example of the latest equipment now (1961) used. A photograph of the dredge which is advertised as the largest in the world appears elsewhere.[10] It contains a pump with a 30 in. discharge and 36 in. suction. The pump is powered by a 6,000 hp motor with a top speed of 425 rpm. There are several dredges of this class with top speeds of 360 rpm. However, the operating speed is generally about 300 rpm. The low output is due to the excessive speed of the pump impeller, which causes cavitation.

Cavitation de-stabilizes the pump performance and makes it impossible to keep the pump loaded to its full capacity. A poor showing when the dredge is first put into operation is generally blamed on an inadequate cutter and mouth piece, improper size of suction pipe, and the kind of soil deposits. The true cause is cavitation.

The pump design technicians have never recognized the importance of the peripheral or tangential velocity of the vane edge as it sugurs through the water in the pump eye. A most unusual phenomenon occurs here which generates water vapor and a bubble train, starting almost at the vane tip. This alters the normal flow distribution in the regular channels. In pump designing due allowance is made for the impact and entrance losses, but this is not directly related to the cavitation. The insert in Fig. 14 shows graphically the conversion of a high impact pressure decreasing to a low pressure on the impeller vane edge. There occurs a snap like action, a phenomena which affects the normal pump operation and chokes off the pump discharge.

The Standards of the Hydraulic Institute endeavors to classify the type of pump in accordance with the NPSH criteria by a back door approach, rather than going directly to the place where cavitation begins.

When the manufacturer develops a new pump which is too large to test in the laboratory, the normal procedure is to make a prototype or model and test the same, and by the established rules determine the operating characteristics of the large pump. If this had been done in the beginning with the large pumps, it is a foregone conclusion that the 30 in. pump would be limited to approximately 195 rpm, unless a way was found to overcome the handicap of excessive speed. The model would have shown the questionable cavitation characteristics.

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[a] February 1961, by Ole P. Erickson (Proc. Paper 2729).

[9] Cons. Engr., Berkeley, Calif.

[10] Engineering News-Record, McGraw-Hill Book Co., New York, December 20, 1951.

From a practical standpoint the large pump can be made strong enough to be safe, and will cost but a small amount more than the slow speed pump. If all the large pumps have the same defect there is no incentive to strive for efficiency. The loss is taken care of by adding to the contract price.

Cavitation is due to two factors. First is the excessive speed of the impeller at the vane tip. For example, when the speed on the vane edge reaches 46.5 fps, vapor bubbles start to form, and this displaces the water in the impeller channels and lowers the discharge pressure. The 30 in. impeller at 360 rpm develops a 34 ft vacuum. This phenomena can be accounted for by using the basic Pitot cylinder formula to determine the pressure imposed on a surface by impact.

The second factor is the suction lift which is limited to 34 ft, or atmospheric pressure. This suction lift plus the speed factor in terms of feet, when added

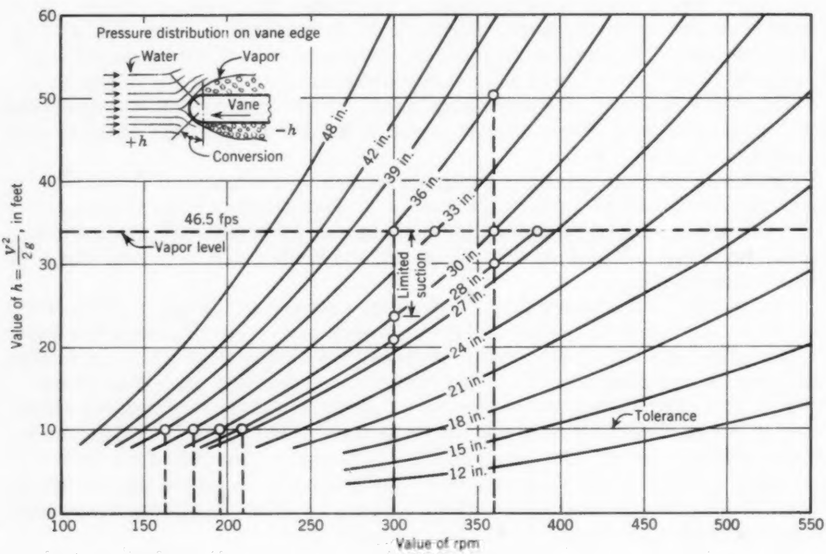

FIG. 14

determines the beginning of cavitation. When the sum is greater than 34 ft, the pump output declines and no one is able to explain why.

There were four pipe line dredge units used at the Fort Peck Dam which were directed by the War Department, Corps of Engineers, from 1934 to 1939. Each unit consisted of an excavating barge on which were two pumps operating in series. All were of the same size, including three additional boosters, 20 pumps in all. The pumps had a 28 in. discharge and the same size impeller inlet. They were powered by a 2500 hp motor with a top speed of 257 rpm. It is significant, as will be evident later, that the normal operating speed of the number one pump was about 220 rpm. The pump had been brought to a high

efficiency through many years of practical operation. The efficiency was probably in excess of 75%. Compare this with the do-it-yourself impeller made from steel plates and by welding, with a questionable efficiency of 60%. There is a difference of 25% in the use of power, and in addition there is greater cavitation due to the crude design.

Refer to the accompanying cavitation data. The 30 in. impeller at 360 rpm develops a full vacuum of 34 ft, which means vaporization. If 24 ft suction lift is required the speed must be reduced to 195 rpm. If the impeller diameter is 36 in. the speed is reduced to 165, to produce the same suction lift.

Referring again to the United States owned dredges, the Department in 1934 reported 40 pipe line dredges in use in sizes from 10 in. to 30 in. The one dredge with a 30 in. discharge ran at 172 rpm, all four of the 24 in. ran at an average for the lot of 205 rpm, and 15 of the 20 in. revolved at 180 rpm. All these pumps will be operated in the low cavitation area of less than 10 ft. This is true of the 28 in. pump at Fort Peck. Had this pump been provided with a 5,000 hp motor, the speed would have been greatly increased. At the low speed the pump served as the excavator with a low cavitation rating and provided the booster with a positive intake pressure, which of course should always be in excess of 15 psi to avoid a suction lift or vacuum. It is no surprise to see a statement that the United States dredges have shown no signs of cavitation in the past 20 yr. Invariably the men in charge of the dredge will deny there is cavitation of importance. They have in mind extensive metal deterioration, noise, and not output performance.

When the operating data for the pump is taken from the records, and the cavitation rating determined by the NPSH criteria, one is led to believe that a suction lift of 25 in. is permissible. This is not correct in the case of the high speed pump. A model test would have made this clear.

The author mentions the automatic relief valve which reduces the "choke-off" from high vacuum, and greatly reduces water hammer in pump and pipe line. This is the Hofer Patented Control which the dredging industry has reluctantly adapted to avoid costly interruption. The necessity for this improvement during the past 10 yr has been due to excessive cavitation.

Now that the industry has been committed to the use of the high speed pump, hampered by cavitation, how can it be improved? It is estimated that only a small amount of water reaches the bubble stage as the bubbles originate just off the extreme tip of the impeller vane edge. A construction that would permit the injection of water or air into this critical zone will improve the operation. A common practice to prevent noise and vibration in a pump is to admit air to the suction pipe. In this case the disturbances quiet down, but the output will will be increased except perhaps through an apparent smoother operation. The output of a pump on the borderline of cavitation, such as the Fort Peck pump, can be increased by jet action.

When 25% to 40% of the shaft energy is dissipated in the pump, the wear will always be terrific. It doubles as the speed increases. The dredge pump has been developed empirically. That is to say, the original design has been padded and the contours changed to provide minimum wear. The mathematical laws governing pump performance were given in the text books 75 yr ago. No one has been able to make a radical change in the pump used for dredging.

The following calculations apply to the vacuum created at the source of origin. It must be kept in mind that the bubble train must diffuse in the impeller channels, and this effect on the pump characteristics cannot be precisely

determined. The greatest need is to eliminate the vacuum at its origin. The point to be made is that there can be an economical limit for increasing the size of the equipment. This can be true of the present 30 in. and larger dredges. This theory of cavitation will account for many of the idiosyncrosies for which there has been no explanation.

EXAMPLES OF CAVITATION RATINGS

Diameter 30 in.

360 rpm	Vacuum 34 ft	No suction lift available.
300 rpm	Vacuum 24 ft	10 ft suction lift available.
195 rpm	Vacuum 10 ft	24 ft suction lift available.

Diameter 36 in.

360 rpm	Vacuum 50 ft	No suction lift available.
300 rpm	Vacuum 34 ft	No suction lift available.
165 rpm	Vacuum 10 ft	24 ft suction lift available.

Diameter 28 in.

360 rpm	Vacuum 30 ft	4 ft suction lift available.
300 rpm	Vacuum 21 ft	13 ft suction lift available.
210 rpm	Vacuum 10 ft	24 ft suction lift available.

A 60-in pump has a capacity of four 30-in. pumps. It is conceivable that in the distant future the 60-in. pump and pipe line can be used, for example, on a desilting project as a booster where the fluid is delivered to the pump at a positive head by slow speed pumps. Cavitation would affect only the pump on the barge.

A pump with a low cavitation rating permits a higher pipe line velocity and, hence, an increased output. A pump with an early "cutoff" due to cavitation such as the 30-in. pump class looses part of its advantage which should accrue to the large units.

In considering the excavation at depths of 200 ft, it must be kept in mind that the margin of hydraulic losses which include entrance, velocity head, friction and the extra weight of the solids is but 34 ft. The limited suction lift can be augmented by a jet propulsion method installed at intervals on the pipe.

United States Patent No. 2,918,017, December 22, 1959, relates to the application of the jet to eliminate cavitation at its origin.

Patent No. 2, 718,717, September 27, 1955, relates to the method of elevating solids in the suction pipe.

The United States Patent Office records several novel methods for excavating the solids.

OLE P. ERICKSON,[11] F. ASCE.—The writer read with interest C. E. Behlke's description of Dutch dredging practice and comparison of Dutch and American type dredges. The writer's company has designed and had built in Holland two American type dredges, the 24 in. discharge, portable Dredge Port Harcourt and the 27 in. discharge Dredge Barbados for Dutch-English joint interest clients. These dredges have all the latest American labor saving devices. The Dutch shipyards do exceptionally good work in dredge building.

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[11] Pres., Erickson Engrg. Co., Tampa, Fla.

The Dutch dredging contractors are rapidly accepting the American dredging equipment, particularly for foreign competition, in the Caribbean, South American, African, and Far Eastern areas.

J. B. Herbich mentions the hopper dredge General Moulthrie as the first dredge using the hydraulic or suction principle. The writer, in his research some years ago, found that two Germans, Hoffman and Schwarztcoff, also in 1855 developed and did use a hydraulic dredge for land filling and canal dredging around Berlin. This dredge had a regular pontoon line with leather sleeves. They also developed and used a cutter.

Herbich mentions portable dredges. The writers' company has designed many portable dredges. The most notable, the 30 in. Dredge Western Chief, 24 in. Port Harcourt, 24 in. Western Hunter and Western Warrior, and a great many smaller portable dredges, both for this country and abroad. The cost of a portable dredge runs approximately 5% over the non-portable type. The strength and sea-worthiness is the same, if properly designed.

Herbich in his analysis of impeller vane angles mentions the hopper dredges, Zulia and the Essayons; these have low head pumps. On high head dredge pumps with pressures ranging from 100 lb to 150 lb, and higher, and discharge velocities of 16 ft to 22 ft, it has been found that the average vane discharge angles vary between 20° to 30°, and entrance angles 1.15% to 1.25% less.[12] Pumps with these, or reasonably close thereto, impeller angles, seem to have less cavitation at high velocities and high vacuum. Model tests also confirm that cavitation is less with these angles.

Herbich asks for clarification as to whether the 40% solids are by weight or volume. The accepted standard for payment, to the writer's knowledge, is by volume, both by the United States Corps of Engineers and private contractors, for harbor, channel, fill dredging. For instance, a 24 in. discharge dredge will pump 3,000 cu yd per handling 40% solids at 18 ft velocity in soft materials, or a mixture weight of 80 to 86 lb per cu ft. depending on type of material.

A. H. Sorensen states that fully lined dredge pumps are used mostly in extremely abrasive materials. The writer knows of many applications of fully lined pumps (some designed by the writer's company) in use both in the United States, Canada, and abroad. There appears to be no valid reason why fully lined pump cannot be used in soft material, and several large all purpose dredges, 30 in. and down to 20 in. have lined pumps, both here and abroad.

As to Sorensen's statement that lined dredge pumps are less economical than conventional pumps, the writer agrees that the first cost of a fully lined pump may be higher, but afterwards complete liner changes can be effected much faster, and of course, the shell or casing liner costs are only a fraction of a new pump casing. Possibly Sorensen refers to sectional plate casing liners, bolted into the casing, which, of course, are slow in changing and expensive, because they have to be fitted in place, and also are difficult to keep tight.

In the writer's opinion, a most important point is the fact that with the present day high production and pressures of 150 lb or higher, a cast pump casing can only be allowed to wear out about 50% to 70% on account of the danger of splitting the casing. With a lined pump the casing liner can be

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12 "Centrifugal and Axial Flow Pumps, Theory, Design and Application," by A. J. Stepanoff, John Wiley and Sons, Inc., New York, N. Y., 1948, p. 97.

allowed to wear until holes appear, which often happens. Also, the shell liner may be made of abrasive resisting materials that will outlast a cast steel pump casing 3 or 4 to 1. This is where the real savings develop.

Sorensen states that it is not practical for a 10 in. pump and pipeline to absorb 600 hp. The writer disagrees. There are several dredges, some designed by the writer's company, that use 600 hp and more on the dredge pump. It is not unusual in heavy material operations to carry 120 lb to 130 lb pressure at 18 ft to 20 ft velocity, to obtain production. A 10 in. dredge with only 200 hp to 300 hp on pump is not very competitive.

The writer agrees that the built-up cutter cost is lower than the solid cast cutter. The writer does not agree that the long time operations favor the one piece cast cutter; nor are these cutters common. On a recent trip through Europe and Japan the writer found that the built-up or cast weld cutter was predominant, particularly American designs, some by the writer's company. The reason is better castings of hub and blades. Renewable wear edges, of various types depending on materials dredged, may be bolted or tack welded, or both, to blades. The writer estimates that probably 75% of the cutters now in use in the United States and Canada are of cast weld construction.

At least two large gear manufacturing companies, which also build cutter reduction gears, follow designs of the writer's company, placing the thrust bearing inside the casing to reduce cost and maintenance. The writer agrees with Sorensen that it is costly to repair the thrust or other reduction gear bearings. They are all liable to fail occasionally, but to the writer's knowledge no "in built" thrust bearings have failed on large dredges.

There are now three large dredges, two 24 in. and one 20 in. using the direct pipe cutter drive, and there are several smaller sizes, two 12 in. dredges, which are capable of dredging to maximum depths of 200 ft. Except for the first 20 in. dredge where some difficulty was encountered with the drive, less trouble has been had than is normal to most conventional dredges. Several more designs have already been furnished to various contractors in the United States, Canada and abroad.

As to Sorensen's statement that there is no proof of less water vacuum with the direct suction pipe cutter drive than the conventional, the entrance losses are less with a round and slightly flared suction opening than with an elliptical, and most times flat opening under the ladderhead. The rotating pipe keeps the material in complete suspension whereas, in the fixed suction pipe, the material often settles to the bottom, creating increased friction. The writer, when first noticing this lower vacuum, had to review the design to discover the reasons. Another feature not expected was less wear. One dredge pumping approximately 3,000,000 cu yd of abrasive sand showed negligible wear on the suction pipe, whereas two shorelines wore out. The writer's explanation, possibly wrong, is that the bulk of material stays at or near the center of the pipe, or possibly more uniform wear, or both.

Sorensen referred to the 40% solids, whether they are by volume or weight. The writer directs him to the preceding clarification of this subject. It may be further said that up to 80,000 cu yd, per 20 hr day, or 4,000 cu yd per hr, or even higher, is common with 27 in. to 30 in. dredges, in soft materials, and it would obviously be impossible to obtain this production by 20% volume.

The writer does not agree with Sorensen's statement that hydraulic dredges cost up to $600.00 per hp, even with average attendant plant included. The initial

cost as stated in the paper does not include attendant plant, which varies greatly for various projects. On the basis of Sorensen's figure, a recent 30 in., 15,000 hp steam turbo electric dredge would have cost $9,000,000.00, which is a shocking figure even in these days.

The writer deeply appreciates A. L. Collins' analysis of pump cavitation and is in complete agreement on this most serious and difficult problem. It has long been recognized by most dredging contractors but little has been done, or can be done about it without greatly increasing pump cost.

Collins mentions the Dredge "Western Chief." This dredge was designed as a portable dredge for a special job, dredging rock, sand and gravel, and to effect closure of the river at Fort Randall Dam. On this dredge, designed by the writer, space and weight were of prime consideration, as it is on most portable dredges. Therefore a balance had to be made against some loss in pump efficiency. The "Western Chief"'s pump top speed was 425 rpm, but it operated mostly from 350 rpm and up, depending on the length of line and type of material handled at the dam. The specific speed was then between 1,000 and 1,100. A larger and slower speed pump has since been installed, reducing the specific speed to between 800 and 900.

The writer agrees that models or prototype tests should be made of all larger pumps before being built, but even then, where the findings definitely indicate a larger pump other factors may decide for the smaller, high speed, cheaper pump. Nowadays with pump pressure ranging from 150 lb to 180 lb, as is often required, a 250 rpm to 300 rpm pump would require an impeller 7 ft to 8 ft. The difference in pump cost would be between 50% to 75%, and it would also require heavier handling equipment.

The writer agrees that cavitation usually begins at the impeller eye by forming vapor or air bubbles at vane edge. This can be held to a minimum by proper vane angles, and by keeping the inner ends of vanes nearly the same diameter as the impeller eye. The common practice of cutting back vanes at entrance to pass larger solids causes increased cavitation and greatly reduces pump efficiency and lowering of top digging vacuum. However, sometimes this is required to reduce down time of dredge. The cut out vane sections should be restored when normal operation is resumed. Another and possibly better way is to use two vane impellers, particularly on short pipelines and easy pumping materials.

As to the 200 ft depth dredging mentioned—this work is now in progress and going well. A hydraulic booster is used on the suction pipe, which is of the rotating or direct cutter drive type.

MARINE OIL TERMINAL FOR RIO DE JANEIRO, BRAZIL[a]

Discussion by P. Leimdörfer

P. LEIMDÖRFER,[16] F. ASCE.—Some points of view on the chapter dealing with fenders are presented herein. This is done with particular regard to the fact that the author based his computation of the kinetic energy to be absorbed by the fender system on the writer's paper.[7] However, it must be stressed that the aforesaid paper comprised the result of investigations made mainly during the years 1951 to 1955. The speed of development in the field of naval architecture, terminal designs, fender systems, and so forth, therefore, induces some of the statements of the said paper to be revised on the basis of new findings.

It is gratifying to read in the paper that one is to start with the fendering system when designing a jetty and to go on to determine the cost relationship between the proper pier structure and those fender types which may be considered. However, having arrived at this point, that is, the choice of the most suitable fender system, the writer feels compelled to draw the attention of harbor designers to the latest findings in the use of tubular rubber fenders. Thus in spite of having dealt with various fender systems (spring-buffers, gravity fenders, dash-pots, the Italian Meca type, and so forth,and disregarding the Raykin buffers, which seem to be a most promising novelty, the writer found the development in the field of the tubular rubber fenders, with the load applied perpendicular to the axis, rather productive. As a matter of fact one cannot but find it astonishing that, up to now, the valuabel properties of rubber tubes have not been exploited satisfactorily.

Thus, admitting that ship thrusts on waterfront installations are inevitable, the problem is to design jetty or dock structures which include a technically adequate fender system and, simultaneously, grant the most economical soluion taken as a whole. Although it is a truism, it must be stressed that the concept of economy comprises taking into account interest resulting from the capital investment, considerations of the longevity of the structure in question as well as the yearly maintenance cost. The writer met many owners who failed to observe this fact and felt strongly tempted to decide merely with regard to the initial expenditure.

In general, most fender systems consist either of merely the shock absorbing unit itself (for example, springs, rubber tubes, gravity fenders, and so forth) or they are composed of two parts. These are the resilient unit fixed to the waterfront structure (for example, rubber tubes, solid rubber blocks, spring buffers, Raykin sandwiches) and a rubbing strip, consisting of a sort of apron,

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[a] February 1961, by H. W. Reeves (Proc. Paper 2733).

[16] Hd., Dept. of Quays and Docks, Stockholm Harbor Bd., Stockholm, Sweden.

sheet or mattress usually of hardwood combined with steel joists or, alternatively, a conventional timber pile and wale system. The purpose of applying the strips or of the pile-wale system is threefold. Primarily, they are to protect the resilient unit from direct contact with the ship's hull side; secondly, they are to distribute the impact forces; and thirdly, they are to enable a smooth berthing operation in the common case of the vessel striking the jetty at a certain angle. Contrary to the author's view, the writer experienced that tubular rubber fenders did an excellent service for several decades without any timber mattresses or pile-wale structures, which, in fact, are indispensable for the majority of other fender systems.

Even in the case of using the finest quality of hardwood damage to the rubbing strips or mattresses cannot be eliminated and almost constant repairs are necessary. On the other hand, according to the writer's observations the result of many years wear and tear on tubular rubber fenders was, as a rule, hardly noticeable provided that they were of suitable quality and were suspended in a correct and purposeful way.

Table 3 illustrates some of the results which aim at throwing light on the economy of different sizes of tubes. Thus the ratio $e = \frac{\text{kinetic energy}}{\text{rubber volume}}$, here called utilization rate, depends principally on the ralation between the outer and inner diameter $\left(\Delta = \frac{OD}{ID}\right)$. Of course, the values of Table 3, can only be approximate because the manufacture of the tubular fenders is not a process furnishing mathematically exact results. Slight differences in regard to the OD or ID and of the circular shape of the tube, as well as any small variation in the hardness of the rubber may influence the said values. Nevertheless, the table fills rather well its purpose because it permits a comparison of the various types A-D of tubular fenders. It is stressed that the influence of supporting chains is not considered in Table 3.

The utilization rates of the table show that economy demands the application of rubber tubes with a ratio $\Delta > 2$. Unfortunately, $\Delta = 2$ seems to be almost standardized by some rubber manufacturers.

It should be stated that any kind of rubber material is affected by the phenomenon of ageing. This means that after a number of years the rubber surface has hardened and the rubber's resilience has diminished.

Therefore, it is necessary to consider the specific ageing factor of the rubber used and then to make an allowance when considering the energy absorbing capacity of the fender chosen.

When using aprons or mattresses it is often desirable to have a rubber fender with one or two plane surfaces. Instead of the rectangular type it proved useful to modify the tubular type by the cutting away of one segment, or even two parallel segments, of the circular section.

Generally speaking, it was always considered one of the disadvantages of the tubular rubber fender that it originated considerable horizontal forces perpendicular to the jetty face. As a result this particular fender system was often overruled since it necessitated the use of additional raker piles or other stabilizing devices in order to grant the necessary factor of safety for the structure.

Yet, to a certain extent these forces may be reduced. Thus, the capacity of the tubes to absorb kinetic energy is, as a rule, proportional to their length as is also the horizontal force. This statement, however, is correct merely

TABLE 3

Type of tubular fender	OD x ID, in inches (mm)	$\Delta = \frac{OD}{ID}$	Area A, in sq in. (sq cm)	Compression of 50%			Compression of 60%			Compression of 67%		
				E_o[a]	$e = \frac{E_o}{A}$[b]	%	E_o[a]	$e = \frac{E_o}{A}$[b]	%	E_o[a]	$e = \frac{E_o}{A}$[b]	%
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
A_1	13 1/2 x 6 3/4 (342.9 x 171.5)	2	107.4 (692.6)	2910 (1.32)	27.09 (0.191)	100	7826 (3.55)	72.87 (0.513)	100	13668 (6.20)	127.26 (0.895)	100
A_2	13 1/2 x 4 1/2 (342.9 x 114.3)	3	127.3 (820.9)	4453 (2.02)	34.98 (0.246)	129	10317 (4.68)	81.04 (0.570)	111	16645 (7.55)	130.75 (0.920)	103
A_3	13 1/2 x 3 3/8 (342.9 x 85.7)	4	134.2 (865.6)	5930 (2.69)	44.19 (0.311)	163	14925 (6.77)	111.21 (0.782)	153	25132 (11.40)	187.27 (1.317)	147
B_1	15 x 7 1/2 (381.0 x 190.5)	2	132.5 (855.1)	3858 (1.75)	29.12 (0.205)	100	10251 (4.65)	77.37 (0.544)	100	19400 (8.80)	146.42 (1.029)	100
B_2	15 x 5 (381.0 x 127.0)	3	157.1 (1013.4)	5688 (2.58)	36.21 (0.255)	124	13338 (6.05)	84.90 (0.597)	110	25243 (11.45)	160.68 (1.130)	110
B_3	15 x 3 3/4 (381.0 x 95.2)	4	165.7 (1068.9)	7275 (3.30)	43.90 (0.309)	151	18188 (8.25)	109.76 (0.772)	142	31085 (14.10)	187.60 (1.319)	128
C_1	16 x 8 (406.4 x 203.2)	2	150.8 (972.9)	4365 (1.98)	28.55 (0.204)	100	11684 (5.30)	77.48 (0.545)	100	21164 (9.60)	140.34 (0.987)	100
C_2	16 x 5 1/3 (406.4 x 135.5)	3	178.7 (1153.0)	6393 (2.90)	35.78 (0.252)	124	15983 (7.25)	89.44 (0.629)	115	28770 (13.05)	161.00 (1.132)	115
C_3	16 x 4 (406.4 x 101.6)	4	188.5 (1216.1)	7716 (3.50)	40.93 (0.288)	141	19687 (8.93)	104.44 (0.734)	135	34612 (15.70)	183.62 (1.291)	131
D	18 x 9 (457.2 x 228.6)	2	191.1 (1232.7)	6393 (2.90)	33.45 (0.235)	100	11906 (5.40)	62.30 (0.438)	100	22817 (10.35)	119.40 (0.840)	100

[a] Absorbed Kinetic Energy, E_o, ft lb per ft (Ton-m per m)

[b] e, ft lb per ft per sq in. (Ton-m per m per sq cm x 10^2)

on the assumption that the fender is applied in a single vertical plane, the number of tubes being immaterial. In the case of a design which has the tubes placed in several vertical layers behind each other, one may design the fender system in such a way that the horizontal forces developed by one single layer of fenders will hardly be increased in spite of a large increase of absorbed energy. Hereby, the original ratio $\frac{\text{energy}}{\text{force}}$ can be considerably augmented. In several designs worked out by the writer for large oil companies and foreign port authorities, energies up to 300 ton-ft could be absorbed by a single group of tubular fenders applied in two or three layers and placed in a recess of the concrete front beam.

It may be mentioned that the amount of 111 ton-ft of kinetic energy calculated by the author for the Rio Terminal could easily be absorbed by using a group of tubular rubber fenders of 16 in. by 5 1/3 in. placed in two layers one behind the other with, preferably, a hardwood mattress between. The total length of these tubes would be approximately 80 ft with a total induced load of about 400 tons. Alternatively, three layers of 12 in. by 4 in. tubes with a total length of 66 ft could absorb the same transmitted kinetic energy with a total load of about 270 tons.

In both cases the curvature of the ship's side was considered when calculating the eventual contact surface.

In principle, the writer introduced the use of tailor-made tubes resulting from the computation that gives the most economic solution with due consideration to the actual dimensions of the suspension units.

To enable manufacturing tailor-made rubber tubes, it is deemed advisable to follow the modern process of wrapping thin rubber lamells around a steel cylinder which is then put in a galvanizing chamber and heated to a high temperature. This way of production secures exact dimensions, which are not always obtained when extruding the solid rubber mass trough, a mouthpiece in conformity with the old method of manufacture.

The supporting system of tubular rubber fenders is of the greatest importance since many failures could be ascribed to the wrong design of it.

The most frequently used horizontal rubber units, placed in the shape of a garland, are safely suspended by high-quality chains whose dimensions grant satisfactory allowance for wear and tear as well as corrosion. The endlinks of the chains lead to shackles which are supported, often by means of strong rings, by a carefully shaped cantilever placed into a recess of the front beam. Alternatively one may place heavy-duty steel tubes right through the front beam and pass the chains through the tubes securing their ends with anchor bolts at the back of the beam.

Vertical rubber units may be supported in a similar way. However, the shackle should be connected with a swivel which enables free turning of the tube around its vertical axis. Moreover, the tube itself may, suitably, be supported at its lower end by a massive rubber bead, through the hole of which a high quality steel bar with a large plate and locking-nut constitute the supporting device. The steel bar passing through the bead may be joined to the chain within the cavity of the aforesaid tube by welding. The resilience of the tube and that of the supporting sphere are to be coordinated.

The use of end plugs and supporting wires, recommended by many rubber fender manufacturers is, in general, to be condemned. The writer had ample opportunities of experiencing the detrimental consequences of these details.

As previously stated, the horizontal load resulting from the use of tubular fenders has, for a long time, discouraged harbor engineers from applying this type of fender. This was the case, too, perhaps at an even higher degree, with the forces transmitted by the longitudinal component of the impact.

In fact, the friction f between the ship's hull and the rubber is considerable. It amounts to f = 0.55 - 0.80 and depends on the ship's surface coefficient as well as on the smoothness, shape, density, and humidity of the rubber tube surface. By applying a pile-wale system or a mattress or apron one may obtain a reduction of f to 0.35 - 0.50.

Another way to diminish the longitudinal forces is the subdivision of the tubular rubber fendering into several layers as mentioned previously.

As regards the adequate dimensions of the supporting chains, consider the following.

a. In the case of horizontal units (-conventional shape of a garland-) at the first thought it seems as if most considerable forces would be originated when assuming f = 0.55 - 0.80. As a matter of fact, however, these theoretically computed enormous values will never come into operation thanks to the initial movement of the tube as soon as the ship's hull touches it. In the next instant, the tube will be pressed to the front beam behind and the longitudinal force is thus transmitted to the jetty structure. Tests showed that the eventual pull acting on the chain will hardly exceed about 10% of the horizontal pressure perpendicular to the jetty front, which sounds reassuring.

b. For the vertically hanging units experience indicated the fact that a loose lower end permitting a swinging motion of the tube is preferable. Apart from the horizontal pull exerted by the longitudinally moving vessel, which often drags the tube with it, a possible rolling motion of the ship is to be considered. With supertankers having a powerful unloading capacity, there is the pull originated by the quickly lowering hull. Observations of several years, and, of course, even failures, proved that the assumption of a total pull of about 15% will be rather adequate for the supporting devices of vertical units.

c. The writer has no experience with inclined tubular rubber fenders (for example, under 45° or 50° to the vertical). However, at some places smaller damage could be seen at the lower support which is probably due to the considerable strain exerted on this point which prohibits axial movement of the fender.

Suggestions for the Rubber Specification.—The fender should be manufactured of the best quality synthetic rubber which should resist the detrimental influence of abrasion, the attack of marine growths and creatures, the aggressive action of salt, oil and acids as well as the alternate exposure to sun and sea water with large temperature variations. An extremely low porosity is demanded to keep the water absorption to a minimum.

The proposal of using natural rubber manufactured with a neoprene cover is not recommended.

Summary.—The writer congratulates the author for his excellent paper dealing, especially with the intricate problem of fenders. This discussion does not aim to make a comparison between different fender systems, the choice of which depends on the site conditions, but intends to give some complementary views on tubular rubber fenders. It is hoped that this small contribution will result in improving the knowledge and increasing the use of the aforesaid fenders in the not too distant future. Furthermore, it is desired that the paper will stimulate the investigation of tubular rubber fenders of greater dimensions having different ratios of the OD to the ID.

GREAT VOLGA WATERWAY[a]

Closure by Otakar W. Kabelac

OTAKAR W. KABELAC,[16] F. ASCE.—The socio-economic concept of the Soviet engineering and the 8-point classification of Soviet river-system development, introduced by Roy F. Besey, F. ASCE, widens considerably the scope of the paper. However, they also provide interesting and important understanding of complex problems, underlying the current development of Russia's water resources, which beside science and technology involve economics, politics, and strategy. These problems in which communication considerations predominate are interlocking, influenced by the continentality of the immense Soviet territory and its inadequate access to ice-free oceans, require solutions on a level different from that of maritime nations of the West.

British geographer Sir Halford John MacKinder,[17] known for his geopolitical theory, in 1904 drew attention of the world to a remarkable distribution of drainage of Euro-Asian landmass; 40% of which is occupied by the present territory of U.S.S.R. Although the drainage accounts for numerous rivers (six of them among the largest of the world), they have been useless for the purpose of human communication with the outer world, emptying into inland salt lakes (Volga, Amu, and Syr-Darya) or into the frozen Arctic Ocean (Ob-Irtysh, Yenisei and Lena). Only 23% of the Soviet territory (8% in Europe and 15% in Asia) is drained into ice-free oceans. The major portion of U.S.S.R. drainage gravitates north, towards the Artics, predominantly under permafrost conditions. This is carried by a system or rivers with an abundant water supply of a combined discharge which exceeds that of all the rest of Russian rivers. The remaining 23% of the Soviet land embraces large areas, depressed in parts below the world sea datum and drained towards inland saline lakes (Caspian, Aral, and Balkhash) which are known for their inherent adverse hydrologic balance[18] (Fig. 9 and Table 2).

It is not surprising, therefore, that the Russian historical trend was directed towards finding a substitute for the inadequate communication between the seas of her domain, and the net of large navigable rivers continually was under consideration as a solution to this problem. The Russian rivers not only played an important role in the economic development of the country on a local level, but they served also as long-distance carriers for the penetration and conquest of vast Asiatic regions, where they represented often the only possible means

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[a] February, 1961, by Otakar W, Kabeloc (Proc. Paper 2749).

[16] Hydr. Engr., Corps of Engrs., Washington, D. C.

[17] "The Scope and Methods of Geography and the Geographical Pivot of History," by Sir Halford John MacKinder, London, 1904.

[18] "Obshaia Gidrologia," by A. I. Tschebotarev, Leningrad, 1960.

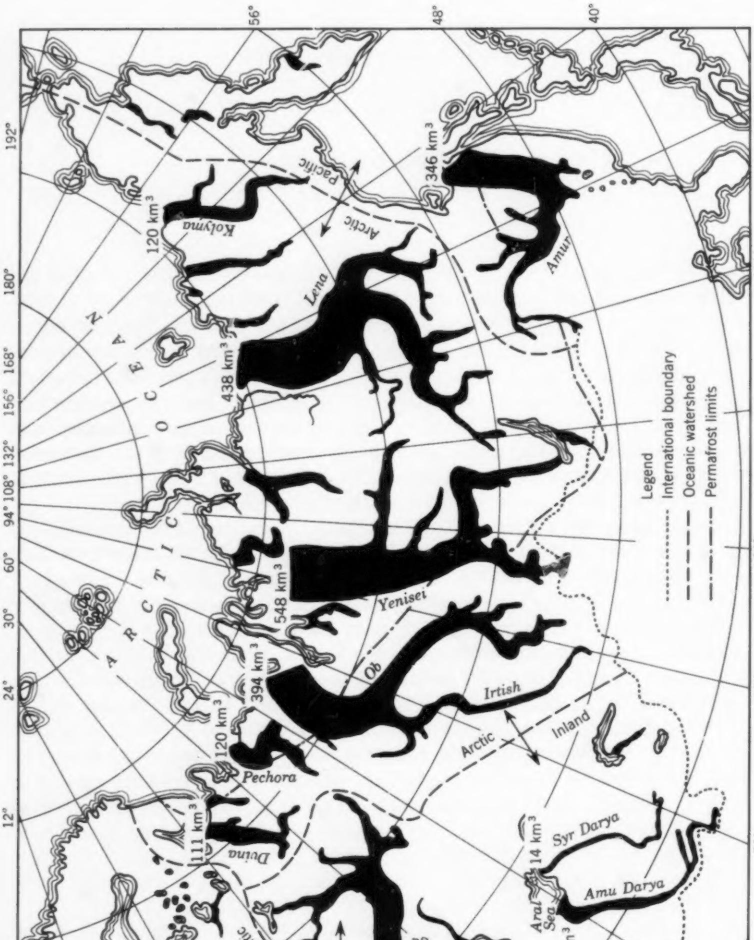

FIG. 9.—HYDROGRAPHIC DIVISION OF THE U. S. S. R.

of transportation. According to statistics, there are more than 100,000 rivers in the U.S.S.R. with a drainage in excess of 100 sq km (38 sq miles). Fifty of them are more than 1,000 km (620 miles) long; 10 are in excess of 2,000 km; and the longest Ob-Irtysh is 4,400 km (2,730 miles). The Volga system alone comprises 7,000 rivers with a total length of 213,000 km (132,000 miles). Out of which 100,000 km are either navigable or can be used for floating timber. According to Zvonkov, the existence of numerous tributaries is an important factor when estimating rivers as means of transportation. In the European part of U.S.S.R., the drainage of rivers are separated usually by a low watershed which greatly promotes the construction of navigation canals linking the rivers in unified systems.[19]

The scientific long-term hydrologic planning and hydrotechnical constructions began in the early stages of Russian communism when the first economic plan GOELRO (Governmental Electrification of Russia) was initiated by Lenin. However, only after a thorough hydrologic survey of available water resources, the basic principles of the planning were formulated.

TABLE 2.—OCEANIC DRAINAGE DIVISION OF U.S.S.R.

Ocean	Drainage			
	in sq km	(sq miles)	in cu km	(10^6 acre ft)
Atlantic	1,800.000	(680.000)	316	(255)
Pacific	3,200.000	(1,200.000)	850	(690)
Arctic	11,700.000	(4,400.000)	2,400	(1,960)
Inland	5,200.000	(1,960.000)	378	(305)

The 8-point classification of Soviet river system development, as defined by Besey, served as guidance for the following remarks:

Overall River-system and Inter-system Scope of the Planning.—Disregarding the propaganda part of the Soviet official reports, it cannot be denied that the world is witnessing the execution of a hydrologic and engineering program embracing a territory of continental proportions and involving a physical reconstruction of the entire Soviet Union. The close analysis of the planning indicates a systematic and total exploitation of water resources by the most advanced methods of science and engineering, which include influencing of hydrologic and meteorologic phenomena. The planning elements include, besides hydropower and irrigation, which in parts supersede in importance other considerations, the promotion of inland waterways as integral part of the Soviet transportation system. The five seas communication concept, formulated at the VIIth Congress of the Communist Party in 1926 and applies to the European part west of Ural Mountains (Fig 10). This development is in final stages of accomplishment, the major components such as Kama-Pechora transfer due in 1965. The XIXth Congress of 1952 included the Asiatic part and river system east of Ural Mountains into the inland transportation unification efforts and great progress has been achieved there.[9] The ultimate unification of European and Asiatic Soviet river systems by the way of the Caspian Sea is in planning. Technically feasible, the construction progress of these plans

[19] "Interaction Between Inland Water Transport and Other Types of Transport in U. S. S. R.," by V. V. Zvonkov, XIX Internatl. Navigation Congress, London, 1957.

adjusted to the required priority schedule in which hydropower and agricultural water supply enjoy considerable preference.[20]

Inter-System Hydrologic Research and Planning.—Following World War II, considerable widening of the program for exploitation of water resources was initiated, and L. K. Davydov was entrusted with overall hydrologic survey which included the Central Asiatic and Siberian parts of U.S.S.R. Davydov showed that there was deficiency of the water supply needed to meet all commitments made in the hydrologic planning, in particular with regard to large irrigation projects in Central Asiatic parts of U.S.S.R. These studies led to the formulation of hydrologic policy, based on total management of water resources, which after Stalin was named "Great Stalin Plan for the Transformation of Nature." The plan is based on the principle "river—creation of climate" provides for "essential changes of the flow and regime of rivers" by comprehensive changes in land surface, correction of watersheds, transfer of river-flows, changes of agricultural cultivation by crop rotation, and reforestation of arid regions.[21] The principles of this policy were manifested in various phases of the Great Volga Project, conducted in the fourth and fifth Five Year Plans during the period 1945-55. The plan also provided the background for the Davydov Plan in 1948.[22]

The major features of the Davydov Plan is the irrigation of the deserts and steppes of the Central Asiatic regions comprising the Kazakh, Uzbek, Turkmen, Kirgiz, and Tadzhik, S.S. Republics by a partial transfer of Siberian rivers south. The plan provided for a navigation system capable of carrying deep-draft vessels from Ob-Irtysh and Yenisei rivers to the Caspian Sea. Approximately 4,000 km (2,400 miles) of waterways are planned; out of which 1,800 km would be inland seas, lakes, and reservoirs; 1,000 km would follow the ancient dried-up riverbeds; and 1,200 km would be newly constructed navigation canals (Figs. 11 and 12).

Inland Navigation in Soviet Economy.—Under the Soviet economy, the waterways are incorporated into a unified transportation system which includes railroads, sea and waterways, highways, airways, and pipelines. All these components, together maintain the communication between various branches of Soviet economy and bind the Soviet territory into an integral economic unit. While the rails carried 81.6% of transportation against 5.4% of waterways in 1958 and the rest divided among other media, the ratio is rapidly changing in favor of waterways and sea transportation. It is estimated that in 1965, the rail transportation ratio will be 73.3% with corresponding increase of other media including waterways. The density of waterway transportation in 1955 amounted to 511,000 ton-km per km, increasing to 865,000 ton-km per km in 1960. In the United States, the corresponding figure in 1954 was 2,550,000 ton-km per km.[19]

The combined transportation of various carriers, in particular water-rails traffic was highly promoted since World War II and is effectively used at present (1961). It took special significance in the European part of U.S.S.R. since the operation of Great Volga Waterway. At present (1961) there are

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[20] "The Importance of Hydrotechnic Construction for the Soviet Transportation," by A. Lebed and B. Iakovley, Translation from the Russian, Munich, Germany, 1954.

[21] "Stalinski Plan Preobrazovania Prirodii V. Deistvii," by B. A. Alexandrov, Moscow, 1952.

[22] "Sviazvanie mezhdu vodami Oba i Aralskogo i Kaspickogo Moria," by M. Davydov, Gidrotekhnicheskoe Stroitelstvo No. 12, 1957.

130 big terminals, such as Moscow, Leningrad, Chelyabinsk, Kuybyshev, Stalingrad, and others. The development and more efficient planning of combined transportation is further fostered by introduction of scheduling for the operation of different types of carriers.[2]

The waterway traffic in U.S.S.R. is not confined to cargo exclusively but extends surprisingly to passenger transportation. Modern passenger liners

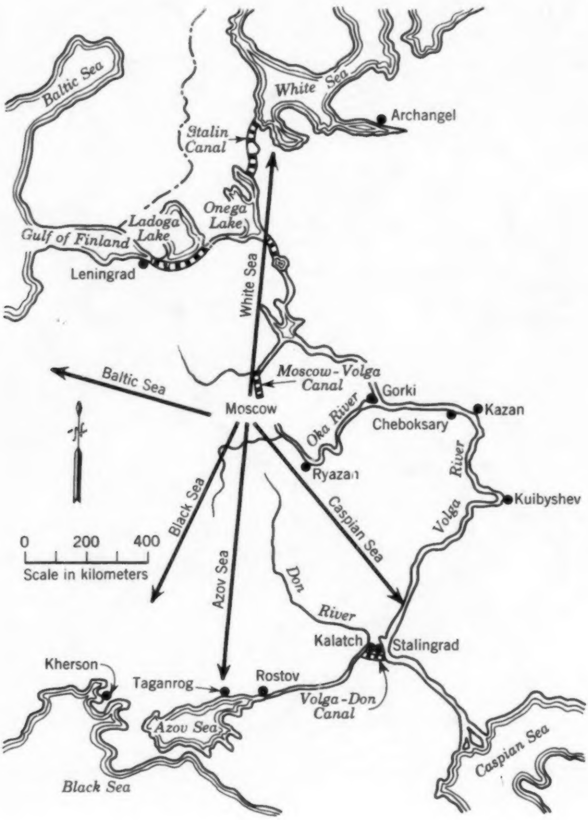
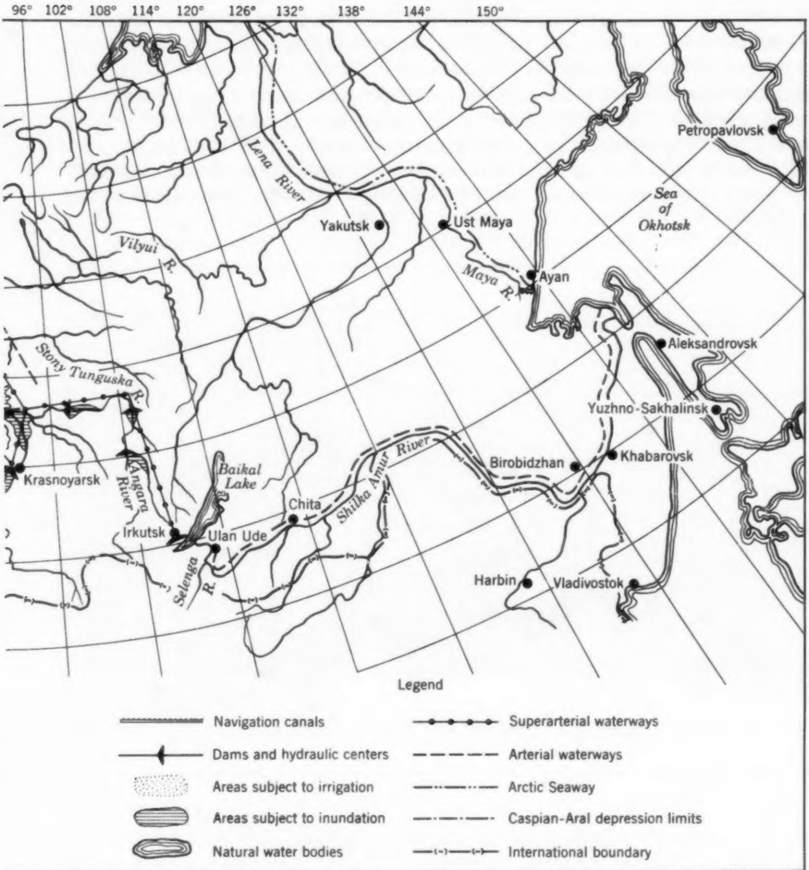


FIG. 10.—MOSCOW-PORT OF FIVE SEAS

are in operation at present between Moscow and Rostov. Fig. 13 represents the newest motor liner (Sovietsky Sojuz) with sleeping and dining accommodations for 486 passengers, which develops an average 16 knots, electrodiesel driven, 2,700 hp. A considerable faster service was introduced recently between Moscow and Gorki, 900 km (560 miles) distance, by diesel speedboats operating on hydrofoil principle. They develop 45 to 50 knots and accommodate

FIG. 11.—DAVYDOV HYDRAULIC SCHEME



FOR TRANSFER SIBERIAN RIVERS SOUTH

sixty persons. Plans are laid for larger and faster vessels of 60-knot speed and 300 passengers, using a hydrofoil principle.[23]

Power Development.—Hydropower production as primary and most urgent component of the Soviet hydrologic planning was initiated in 1921 by the GOELRO Plan of Lenin. It was presented by Soviet propaganda as a magic cure for weaknesses of Russia, based on Lenin slogan, "Communism is the Red Army plus electrification of our country." The majority of hydropower producing schemes involved then were developments on river headwaters in mountainous regions (Ural, Caucasus, and Pamir) by use of high valley dams (Talsperren) and high-pressure turbine units. This type is still being constructed for marginal local production. The general trend, however, is towards the power production in hydraulic centers, where power units are placed in the

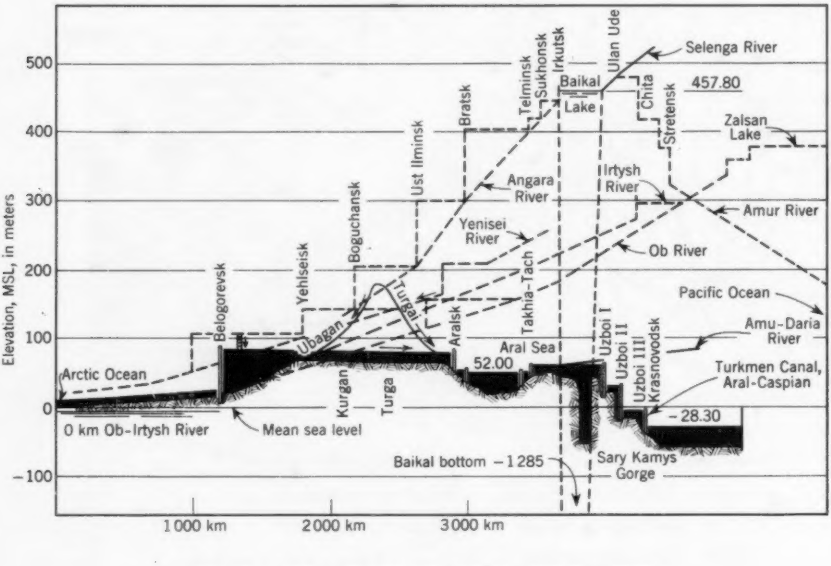


FIG. 12.—DAVYDOV PLAN ELEVATION SCHEME

main stem of large rivers, such as described for the Great Volga. The present Seven Year Plan 1959 to 1965, emphasizes the hydropower potentialities of the Ob-Irtysh and Yenisei-Angara rivers, including the gigantic water supply reservoir of Lake Baikal Angara river, tributary of Yenisei is evaluated by 60,000 mw of potential energy with 10,000 mw under the present Plan development. The largest center under construction at Bratsk, will have installed production capacity of 3,200 mw, higher than Kuybyshev or Stalingrad. It is planned for operation in 1965.

Irrigation Development.—Irrigation and rehabilitation of steppes and arid regions for agricultural cultivation or reforestation, and the development of areas needed for resettlement of population on a large scale, besides providing

23 "Rechnoi Transport U. S. S. R., 1917--1957," Moscow, 1957.

water supply for a growing number of large industrial centers are other major components of the Soviet hydrologic planning. Flood protection and reclamation works are constructed where feasible. In numerous instances navigation canals serve simultaneously for irrigation and drainage of large melioration projects, such as described for the Great Volga. The Davydov Plan, mentioned previously, contemplates irrigation of 250,000 sq km (96,500 sq miles) and additional reforestation of 200,000 sq km (76,000 sq miles) of Kazakhstan, S.R., by diversion of 315 cu km (Figs. 11 and 12) (255 by 10^6-acre ft) per yr from Ob-Irtysh and Yenisei, south. Out of this volume, 75 cu km (60 by 10^6-acre ft) would serve for revitalization of Caspian Sea. This increase of cultivated and forested areas would influence considerably the climatic conditions of the entire region and inprove its hydrologic adverse balance.[22]

Stress on Multiple-use and the Comprehensive in Overall Planning.—On the XVIth International Congress of Navigation in Brussels in 1935, the Soviets

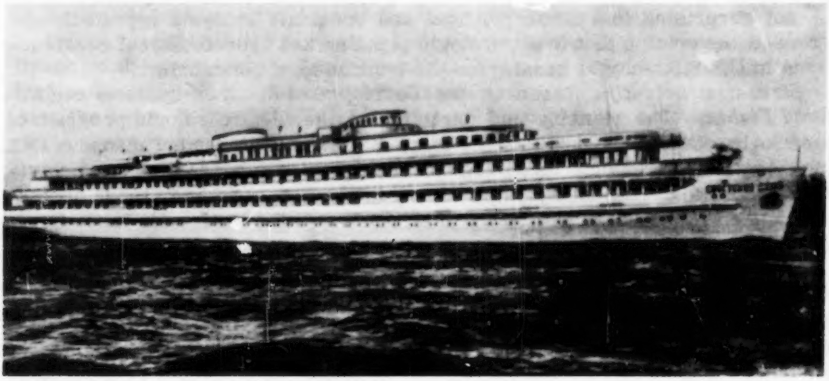


FIG. 13.—LINER SOVIETSKY SOJUZ ON MOSCOW-ROSTOV RUN (2,700 HP ELECTRO-DIESEL)

indicated for the first time the trend that they were going to follow in the development of waterways and that of Timonoff, Stalin's advisor, on transportation named "Maximalization of Rivers." In his paper presented to the Congress, he said:

> "The maximalization of rivers has as its starting point as complete as possible a use of hydropower not only for industrial purposes but also for the purpose of changing the hydrography of the county. The electricity generated by the dams will serve in the case of need to lift the water and make it run in new direction. The combination of the two developments open perspectives on which we at present place no limits. We shall be able to connect distant rivers by new waterways, rivers with watershed inaccessible to undertaking of this kind. We shall be able to supply water to localities that have been considered doomed to perpetual penury. We may perhaps make the deserts habitable and prosperous. The change of flow of rivers will require large canals so that navigation may be carried on.[5]"

The Soviets have found the utilization of rivers, based on the maximalization theory, along with the promotional and decentralization effects of waterways of paramount importance for the development of the vast territory of the Union. The multiple-purpose hydraulic development, known as "Gidrouzel"—Hydraulic Center represents a standard form for this purpose. They control extensive river reaches and combine in one place flow regulation, power production, and navigation. Construction progress is nearly always such that navigation facilities may be added when priority schedule calls for power production preference. They serve large regions through long-distance high-voltage power distribution grid. These hydraulic centers are duplicated in great number on all major rivers, attaining gigantic proportions on the Great Volga, Ob-Irtysh, Yenisei—Angara, and others. The Soviets consider the hydraulic centers as prime movers in the economic mechanism activated by water falling on the river-basin area. Besides water supply, they provide for the basic economic elemets such as power, transportation, food, and raw-material production and may ultimately be used for influencing of hydrometeorological phenomena. It is not surprising that some political and economic analysts repeatedly expressed the opinion that the hydrologic planning and hydrotechnical constructions in U.S.S.R. may be considered the foundation of communism.[9]

Stress on Scientific Research and Development in River-systems and Related Fields.—The planning and execution of the vast hydrologic program of the Soviet Union could not be conducted without a competent manpower. A high regard for science technology and engineering along with special privileges, both material and social, attached to persons engaged in those professions accounts for a large number of students seeking careers in those fields despite a rigid discipline applied in education. Recent statistics on the number of graduating engineers in U.S.S.R. compared with United States brought up in United States press confirm this statement. Under the highest government sponsorship, a wide interest is being promoted in hydrology, hydraulics, meteorology, geography, geology, and related disciplines dealing with natural water-cycle phenomena. Research in those fields, implimented by laboratory investigations and expeditions in many parts of the world meets the highest scientific standard, progressing constantly into new and diversified fields. The volume of scientific and technical literature has constantly increased since 1928, the beginning of the first Five Year Plan, with unusual advance since 1946. The professional literature consists of books, pamphlets, periodicals, and papers published by institutions such as Academy of Sciences of U.S.S.R.; also by academies of individual Soviet Republics, such as Ukrainian, S.S.R., Kazakh, S.S. Republic, and others. Engineering and technical magazines in diversified fields offer a great opportunity for writing, in particular for young adepts of science and engineering professions and students. Variety of textbooks provide for professional education in sciences and engineering on all levels.

Other World Approaches to River Development.—The period following World War II, dealing with economics resulting from dislocation and territorial changes brought up a considerable reevaluation of promotional role of rivers and waterways in modern economic planning of large geographic areas, Several German and French publications and papers indicate the trend prevailing at present in the thinking of political economists and transportation engineers.[24,25]

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24 "Wasserstrassen und Raumplannung," by Karl Foerster, Bonn, Germany, 1956.
25 "Les Hydrostrades de l'Avenir," by George Hersent, Paris, France, 1950.

The XXth International Navigation Congress in Baltimore September 11 to 19, brought up for discussion "criteria for the economic justification of new inland navigable waterways or improvement of existing ones" as Subject 1, Section I-Inland Navigation. Great number of papers presented at the Congress and conclusions taken indicate a world wide interest in waterways effects on creation of new economic activities.[26]

The progressively growing demand for power in the newly developed areas and territories, together with the requirement of water for industrial, agricultural, cities supply, and numerous other purposes makes it mandatory, in most cases, to exploit the water resources of the country to its limit. Since 1900, scientific achievements in the field of hydrology, meteorology, geology, soil mechanics, and other disciplines of geophysics, also progress in river engineering and hydraulic structures indicate the way towards the most efficient solution. It calls for a planned hydrologic and hydrotechnical development of an entire river basin or combination of basins. For large rivers extending over a area of divided sovereignty, an international agreement for a joint exploitation of the water falling on the area of the basin, is necessary.

Since World War II, there have been several large hydropower-navigation international projects; the Saint Lawrence river development, based on United States-Canada international agreement being the most outstanding example. Also the India-Pakistan Indus river joint development and the German-French Rhine river reconstruction indicate the way towards international cooperation in water resources exploitation.

The Soviet Union in her effort to strenghten the ties with her satellites, since 1945 took full advantage of the occupation of the Danube basin and established her domination on this important waterway of Europe. Formally the Soviets entered into a multilateral agreement with the satellites, Rumania, Bulgaria, Hungary, Czechoslovakia, and also with the non-satellite Yugoslavia and Austria and established Danubian Commission for a joint exploitation of navigation facilities of the Danube river from Sulina on the Black Sea to Vienna.

The achievements of Soviet engineering in water resources development have produced a following outside of U.S.S.R. It is only natural that the Soviet satellites accept the collective engineering methods and the 5-yr planning in water resources exploitation, besides adopting Soviet advice and using their machines. However, many uncommitted countries of Asia and Africa such as India and Egypt, seek help of the Soviets in their economic and engineering problems.

In this respect the Soviets are greatly supported by their activities within international scientific economic and technical institutions and associations, such as the United Nations Educational, Scientific, and Cultural Organization (UNESCO), International Union of Geodesy and Geophysics (IUGG) and its subdivision of Physical Oceanography (IAPO), Meteorology (IAM), Hydrology (IAH), International Association for Hydraulic Research (IAHR), International Commission on Irrigation and Drainage (ICID), Permanent International Association of Navigation Congresses (PIANC), World Power Conference (WPC), and many others. The Soviets participate actively on meetings and conferences of those international bodies by delegates proficient in their specialized professional fields, and competent to propagandize the scientific and technical progress of U.S.S.R.

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26 "XXth International Navigation Congress, Baltimore-U. S. A. 1961 Section I. Inland Navigation. Subject 1," Permanent Internatl. Assn. of Navg. Congresses, Brussels 4 Belgium.

VARIABLE HEAD TECHNIQUE FOR SEEPAGE METERS[a]

Discussion by Dan Zaslavsky

DAN ZASLAVSKY.[4]—The author assumes a uniform hydraulic conductivity throughout the lining bottom of the reservoir. In a wide range of experiments it was found that this was never the case.Depending on the nature of soil at the bottom of the pond and the history of the pond, the hydraulic conductivity varies in the profile. In swelling clay soil, the top layer of the reservoir bottom was found completely ineffective after a period of time, in forming resistance to flow as compared with the lower layer which formed the main resistance to flow.

In soils with a smaller quantity of clay the opposite is true. The top layer is generally well dispersed and thus less permeable and the layers underneath which retained their structure are relatively more permeable.

It was shown theoretically and experimentally that when the water percolates downwards from a relatively impermeable to a relatively permeable layer an unsaturated flow formed right below the impermeable layer.

It seems that the analysis brought in the original paper would fit only in cases in which the reservoir bed is of stable granular material and of uniform conductivity.

[a] March 1961, by Hermon Bouwer (Proc. Paper 2775).

[4] Lecturer on Soil Physics, Dept. of Soil Engrg., Faculty of Civ. Engrg., Israel Inst. of Tech., Haifa, Israel.

DESIGN AND STABILITY CONSIDERATIONS FOR UNIQUE PIER[a]

Discussion by Palmer W. Roberts

PALMER W. ROBERTS,[7] F. ASCE.—In the stability analysis presented by the authors, one factor stands out by virtue of its apparent inconsistency with an otherwise logical approach to a major design problem. This inconsistency is arrived at by the assumption that the ten wooden fender piles, estimated by the authors to have a spring constant of 7 kips per in. per pile, will reduce the maximum stress on the steel piles from 25.8 ksi to 14.0 ksi. Disregarding any technical design analysis for a moment, one should consider that what the authors are stating is that the addition of ten wooden piles at the periphery of the pier will reduce the maximum fibre stress in 889 steel bearing piles by almost 50%.

The enormity of the contribution of these ten wooden fender piles to the stability of the pier should warrant a thorough analysis of the design concepts involved. It is the opinion of the writer that these fender piles, fastened to the pier deck, will follow the pier in its deflection in about the same manner as the steel bearing piles will. Consequently, they should be considered as working in parallel with the steel piles, rather than in series as the authors have concluded. Next consider the spring factor of these wooden piles. Assuming the piles to have a 10 in. diameter at their point of fixity, they will have a spring factor of 70 lb per in. per pile (rather than 7,000 lb per in.), or 700 lb per in. for the 10 piles.

$$I = \frac{\pi d^4}{64} = \frac{3.14 \times 10^4}{64} = 490 \text{ in.}^4$$

$$E = 1{,}600{,}000$$

$$L = 512 \text{ in.}$$

$$\text{Spring constant } K = \frac{12\ E\ I}{L^3} = \frac{12 \times 1{,}600{,}000 \times 490}{512^3}$$

$$K = 70 \text{ lb per in. per pile (wood)}$$

$$K = \frac{12 \times 30{,}000{,}000}{512^3} \times 326 = 880 \text{ lb per in. per pile (steel)}$$

Comparing the spring constant for the ten wood piles with the 880 lb per in. per steel pile, the effect on the maximum stress of these ten wood piles would be less than would be obtained by the addition of one steel pile.

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[a] May 1961, by James Michalos and David P. Billington (Proc. Paper 2807).

[7] Capt., CEC, USN, Dist. Civ. Engr., 12th Naval Dist.

If it were the intention of the authors to introduce a camel or boom log between the vessel and the fender piles, as is the custom in naval shipyards, some of the kinetic energy would be absorbed by the deflection of the fender piles. However, this could still not justify the author's contention that the fender piles would act in series with the bearing piles. On the contrary, they would be acting in parallel for the following reasons:

1. Because some of the force acting on the fender will be transmitted at the point of fixity, the entire force cannot be considered as being transmitted to the pier.
2. When the fender piles, taken as a unit, are considered as the first spring and are subjected to a force exceeding their elastic limit, they will break. This will interrupt transmission of the acting force to the pier, which is considered as the second spring.

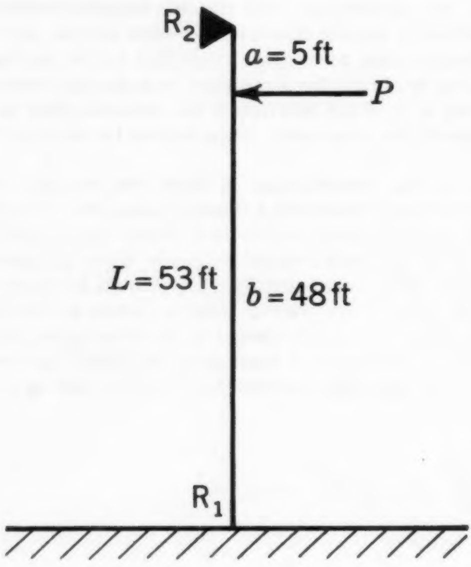

FIG. 9

It is the writer's opinion that when a camel is used an analysis should be made to determine the energy that will be absorbed by the piles in contact with the camel, under stresses up to the elastic limit of these piles. This energy should then be deducted from the kinetic energy imparted by the vessel, and the reaction of the pier should be determined on the basis of the remaining energy to be absorbed.

To determine the energy that will be absorbed by the ten wooden fender piles, it was assumed that the piles had an average diameter at the top of 14 in.; that the point of force application was at the center of gravity of the

camel floating at mean high water, or approximately 5 ft below the point at which the piles were fastened to the deck; that the elastic limit of the wood piles was 8,000 psi; and that the length of the piles from the fastening point at the top of the deck to the point of fixity was approximately 53 ft (see Fig. 9).

$$I = \frac{\pi d^4}{64} = \frac{3.14\ (14)^4}{64} = 1{,}884 \text{ in.}^4$$

$$S = \frac{I}{\frac{1}{2}d} = \frac{1{,}884}{7} = 269 \text{ in.}^3$$

$$M = R_2\ a$$

$$M = f\ S$$

$$R_2\ a = f\ S$$

$$R_2 = \frac{f\ S}{a}$$

$$f = \text{elastic limit} = 8{,}000 \text{ psi}$$

$$a = \text{distance} = 5 \text{ ft}$$

$$R_2 = \frac{8{,}000 \times 269}{5 \times 12} = 35{,}800 \text{ lb}$$

$$P = \frac{R_2 \times 2\ L^3}{b^2\ (a + 2\ L)} = \frac{35{,}800 \times 2\ (53)^3}{(48)^2\quad (111)} = 42{,}000 \text{ lb}$$

$$P = 21 \text{ tons}$$

$$\frac{R_2}{P} = \frac{35{,}800}{42{,}000} = 0.85 = 85\%$$

$$\text{Deflection} = \frac{P\ a^2\ b^3}{12\ E\ I\ L^3}\ (3\ L + a)$$

$$a = 5 \text{ ft} \qquad b = 48 \text{ ft} \qquad L = 53 \text{ ft}$$

Using the preceding values and standard equations, it follows that a force of 21 tons is sufficient to break a pile; the percentage of the total applied force reacting on the pier is 85%; the reaction on the pier is 178 tons; the deflection of the piles at the point of applied force is 0.5 in. and the deflection of the pier under the pressure of 178 tons transmitted by the piles, is 0.44 in. It was further determined that the total energy absorbed by the piles would be 99. in.-tons of the total kinetic energy of 2,020 in.-tons transmitted by the vessel (as derived from the assumptions made by the authors).

In view of the preceding, the best that can be expected from the fender piles is that they provide a cushioning effect between the vessel and the pier, slightly

reducing the effect of the force of 1,690 kips delivered to the pier (determined by the authors in Eq. 15.).

Although the authors indicated in their schematic plan that a 755 ft berth was available at the west end of the pier and showed a single large vessel berthed at this location in the artist's rendering, their analysis did not include any mention of this berth. The approximately 4 in. of deflection that the authors claimed for berthing on the north and south sides of the pier cannot be expected for berthings on the west side. Here the pier will prove much stiffer to bending since the bearing piles are oriented with their major principal axis perpendicular to the acting force. Furthermore, deflection of the pier will be restricted by the physical limitation of space, because the east of the pier is firmly secured to Manhattan Island. Under these conditions the pier is statically stable but devoid of any resilient fendering system capable of absorbing the energy of impact of a large vessel. As a result, continuous damage to the fender piles can be expected, with the possibility of damage to the pier and vessel in the event of a berthing under the extreme conditions assumed by the authors.

The authors did indicate that under certain conditions the fender piles may rupture, but that they did not consider this serious because the piles could readily and cheaply be replaced. The writer does not share their opinion that fender piles can readily and cheaply be replaced. Many years of experience at naval shore establishments has proven that replacement of the conventional pile fender system is neither cheap nor readily accomplished. On the contrary, replacement costs for fender piles has been one of the major items contributing to high maintenance costs at the waterfront, in addition to precluding the use of the berth for a prohibitive length of time while new piles were being driven and fastened in place.

ECONOMICS OF RIVER BANK STABILIZATION[a]

Discussion by Howard J. Mullaney

HOWARD J. MULLANEY,[2] F. ASCE.—The author has outlined quite clearly the many complexities entering into the determination of the economic justification of a bank stabilization project. These complexities arise primarily in estimating the project benefits. Here the engineer is plagued by both the dearth of recorded data and the uncertainties of future conditions. Considering the large measure of judgment that must enter into the benefit determination, the method of discounting the benefits to an equivalent uniform annual series may not appear to be of sufficient importance to warrant extended discussion. But the methods used by the author result in such substantially different answers that some comment appears warranted.

The author states that if the benefits were to accrue to the Federal government the proper rate for discounting them to present worth would be $2\frac{1}{2}\%$, or the same rate as the Federal interest rate used on the cost side of the ledger. However, because the benefits accrue to private landowners he uses a non-Federal rate of 4% to discount the benefits which are then compared to annual costs figured at $2\frac{1}{2}\%$. It would seem that the anomaly he mentions arises from the fact that it is anomalous to assume two differing viewpoints, Federal and non-Federal, in reaching a decision on a proposed Federal investment. It is believed that the Federal viewpoint should be assumed in deciding whether or not to make such an investment. The alternatives that are available in lieu of such an investment need to be considered. Obviously, the investment of Federal funds in the non-Federal sector is not such an alternative and the rate of earning in that sector is therefore not pertinent to the decision.

The alternatives to the Federal investment in bank stabilization include investment in other Federal projects, reduction of the national debt or reduction of taxes. Consideration of these alternatives involves analysis of opportunity costs, that is, what is foregone by making the investment in question. Although such an analysis may indicate that the base cost of long-term Federal borrowing is a minimal rate of return for such investments, this problem goes beyond the point the writer wishes to make: namely, that whatever rate is used, that rate should be used for discounting both benefits and costs. The purpose of such discounting is to compare a stream of dollar costs with a stream of dollar benefits when either of the two streams does not accrue uniformly, taking into account the time value of money. It is unrealistic to assign a different rate to the benefit dollars, and to do so distorts the desired comparison.

The distorting effect of the use of different rates of interest to the costs and benefits is accentuated by the author's "total cost" basis of comparison.

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[a] May 1961, by Charles Senour (Proc. Paper 2809).

[2] Bd., of Engrs. for Rivers and Harbors, Washington, D. C.

Not only is the result illogical, but the difference is much greater than would be expected from the difference in the two rates. Analyzed by present worth methods, and using $2\frac{1}{2}\%$ for both benefits and costs, the project has an indicated benefit-cost ratio of 1.11. Using $2\frac{1}{2}\%$ for costs and 4% for benefits the benefit-cost ratio is 0.97. This change is at least in the expected direction. If the interest rate applicable to the benefits increases, the future benefits would be less attractive and cash on hand more attractive because it can now earn more. Thus, the project depending on future benefits becomes less desirable as the interest rate increases. However, the method of total costs, using $2\frac{1}{2}\%$ and 4% rates for costs and benefits, respectively, results in a benefit-cost ratio of 1.51. Contrary to expectations not only is the project economics improved by this method but it is improved by a great amount. It is the use of the different rates of interest rather than the method that causes the great disparity. If the method of total costs is used, using $2\frac{1}{2}\%$ throughout, the benefit-cost ratio is 1.11% exactly what it was using the present worth discounting method and $2\frac{1}{2}\%$ for both benefits and costs.

In summary, although the rate applicable to long-term Federal borrowing may be inadequate to reflect the opportunity costs of Federal investment, as long as that rate is used for estimating Federal annual costs it should be used also for discounting the benefits accruing to the Federal project.

PNEUMATIC BREAKWATERS TO PROTECT DREDGERS[a]

Discussion by J. B. Schijf

J. B. SCHIJF.[34]—The possibility of attenuating or even annihilating waves by means of a curtain of air bubbles has a certain fascination. This is the reason, no doubt, that after the first attempts of Brasher in the early 1900's, so many studies have been made on the subject. The writer participated many years ago and has followed later publications with some interest. The author deserves our thanks for his extensive survey of all these studies and for his attempt to present them in a comprehensive frame.

In spite of this, the outcome is somewhat disappointing, not only from the point of view of the feasibility of protecting dredgers operating offshore, but also because the picture emerging is not so clear as one could wish. The author has summarized the results of a number of laboratory tests and field experiments in Fig. 1, with the L:d- ratio as sole parameter. Although the L:d- ratio undoubtedly is an important factor in the problem, this seems to be a rather strong simplification and the question arises if perhaps more coherent results might have been obtained on the basis of a tentative analysis of the physical processes. This is not certain because apart from the generally agreed conclusion that an air curtain has more effect on short and steep waves than on long waves, the results of the different investigators show considerable scatter and in some cases are even more or less at variance with each other. Presumably this is to be ascribed to some extent to the fact that most of the studies, in particular the earlier ones, such as the writer's own, have been conducted largely experimentally without much guidance by an understanding of the physical processes involved. A further difficulty lies in the fact that not in every case complete information is given on the relevant data (wave characteristics, length of the curtain, particulars on the system of air injection, depth, pressure and quantities of air, and location of wave height measurements). Moreover, as will be seen, there may be reasons to be wary in some cases about accepting the data at their face value. But, even if a better understanding of the physical processes might not enable to make a better use of the information available at present, it will still be valuable, because it will show the nature of the gaps in our knowledge and it can serve as a guide for possible further research.

The writer believes that an attempt to analyse the physical processes can be made on the following lines. On the strength of the observations made by several authors, the theoretical studies of G.I. Taylor[22] and others and the

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[a] May 1961, by James L. Green (Proc. Paper 2815).

[34] Assoc. Prof. of Civ. Engrg., Princeton Univ., Princeton, N. J.

special experiments of J. T. Evans[6] it seems that two conclusions of a qualitative nature are warranted:

1. The effect of the curtain is almost entirely due to the branch of the current system opposing the oncoming waves.
2. No really significant attenuation occurs until the waves break completely or partially in front of the curtain. The fact that an apparent damping can be observed without breaking will be discussed subsequently.

On the first point most of the authors agree. It seems to be conclusively established by the experiments of Evans,[6] who reports identical effects on waves by a curtain of air bubbles and by a current with the same velocity distribution induced by a water jet.

The second point follows from the consideration that the breaking of waves is much more effective in dissipating energy than any other process that might be engendered by the counteraction of waves and currents. It is moreover supported by the observations of W. Hensen[26] and the writer.[21] Other processes may be contributing, but they can hardly have more than minor importance.

Departing from these two points, the problem can be divided into two parts:

1. The creation of a current system by a curtain of air bubbles, and
2. the action of an opposing current on oncoming waves.

Regarding problem 1, on the creation of a current system by a curtain of air bubbles, rather complete information is available. Several authors present direct measurements of the strength and the velocity distribution of such currents. The experiments of Evans[6] have already been mentioned. Nearly simultaneously with the paper under discussion P. S. Bulson[35] has published an extremely interesting account of a large scale investigation, conducted in the Trafalgar Graving Dock, Southampton.

Both Evans and Bulson have found a practically triangular velocity distribution in the surface currents generated by the ascending air, with a much slower return current below. Fig. 2 shows the velocity distribution according to Bulson.

This means that the strength of the current is entirely defined by the surface velocity and the depth of zero velocity. From his observations Bulson has derived the relationships

$$V_m = 1.46 \, (g\,Q)^{1/3} \qquad (3)$$

and

$$T = 0.32 \, H_0 \log_e \left(1 + \frac{D}{H_0}\right) \qquad (4)$$

in which V_m is the velocity at the surface at a distance D from the axis of the curtain; Q represents the volume of air per unit length per second; T is the depth of zero velocity; H_0 denotes atmospheric pressure expressed as head of water; and D is the depth at which air is injected.

This means that V_m is dependent only on Q and T only on D, at least if Q represents the volume of air under pressure $H_0 + D$. At atmospheric pressure $Q_0 = Q\left(1 + \frac{D}{H_0}\right)$, so that Eq. 3 changes into

[35] "Currents Produced by an Air Curtain in Deep Water," by P. S. Bulson, the Dock and Harbor Authority, May, 1961.

$$V_m = 1.46 \, (g \, Q_0)^{1/3} \left(1 + \frac{D}{H_0}\right)^{1/3} \quad \ldots\ldots\ldots\ldots \quad (5)$$

With small values of D the thickness of the current is nearly $\frac{1}{3}$ D, at 20 ft about $\frac{1}{4}$ D, and at 40 ft about $\frac{1}{5}$ D.

These relationships apply not only to Bulson's full scale experiments (depth up to 34 ft), but also to laboratory tests of Evans (depth 3 ft) and by the British Hydromechanics Research Association, which are mentioned by Bulson. Attention should be drawn to the third power relation between V_m and Q

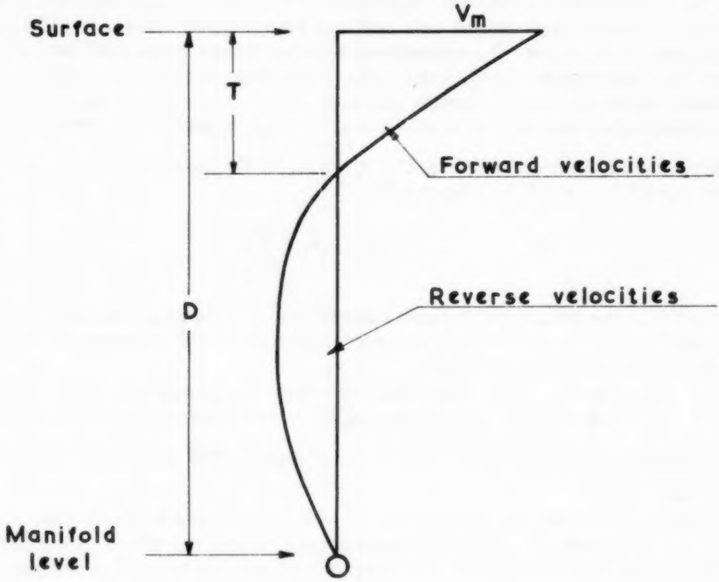

FIG. 2.—VELOCITY DISTRIBUTION ACCORDING TO BULSON

(already predicted by Taylor from theoretical considerations). It means that an increase in V_m requires a much larger increase in Q (and consequently in power).

According to Bulson's experiments it makes no difference if the same quantity of air is passed through different orifice diameters, or if one or two manifolds are used. Several other authors have reported a similar experience. Still it seems to be somewhat curious. The horizontal currents are a result of the elevation of the waterlevel, caused by the decrease in density of the water column above the manifold. This decrease in density is proportional to the volume of air contained in the water. It would be expected, therefore, that small bubbles, having a slower ascending velocity and so remaining longer

in the water before reaching the surface, would have more effect than larger bubbles. Possibly the travel time of the air from the point of release to the surface is determined by the mass effect rather than by the behavior of the individual bubbles. The question remains, however, if for still smaller depths, perhaps around 1 ft, at which several laboratory tests have been made, there is still no influence of bubble size. Additional research on this point might be useful in order to ascertain the validity of small scale tests.

If apart from that the relationships established by Bulson are accepted as valid within a fairly wide range of conditions, and there seems to be no reason why they should not be accepted, one part of the total problem at least seems to be clarified.

Regarding problem 2, it appears to be useful first to consider the effect on waves of a current over the full waterdepth. This is simpler than the case of a current in the surface layers only and can be analyzed more completely. As will appear later, several conclusions can be drawn from such an analysis that are at least qualitatively valid also in the case of a surface current, and for rather short waves the results can even be brought to bear quantitatively.

For deepwater waves, or surface waves, that is waves for which $L:d \lesseqq 2$, the wave velocity is expressed by $C = \sqrt{\frac{g}{2\pi} L}$. In that case it is found that in a current the wave velocity changes to[36]

$$C = \frac{C_0}{2}\left(1 + \sqrt{1 + \frac{4\,V}{C_0}}\right) \quad \text{.............} \quad (6)$$

in which C_0 is the velocity of the undisturbed wave; V denotes current velocity, taken positive in the direction of wave propagation; and C represents the wave velocity modified by the current.

This means in the first place that no waves can penetrate into a region of opposing current having a velocity equal to or above a fourth of their own phase velocity $\left(-V \geqq \frac{1}{4}\,C_0\right)$. Even the lowest waves will break on meeting such a current.

In general however the waves will already be brought to the breaking point by a slower current, because the opposing current has the effect of heightening and shortening the waves, so that their steepness increases. A simple computation yields

$$\frac{L}{L_0} = \frac{1}{4}\left(1 + \sqrt{1 + \frac{4\,V}{C_0}}\right)^2 \quad \text{............} \quad (7)$$

and

$$\frac{H}{H_0} = \sqrt{\frac{2}{1 + \frac{4\,V}{C_0} + \sqrt{1 + \frac{4\,V}{C_0}}}} \quad \text{............} \quad (8)$$

These relationships are shown in Fig. 3 (curves 1). The effect on the wave steepness $\left(\frac{S}{S_0} = \frac{H}{H_0} : \frac{L}{L_0}\right)$ can be taken from these curves. If a steepness of 1 in 7 is taken as the limit for breaking, for waves of different steepness, the stopping velocities listed in Table 8 and presented in Fig. 4.

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36 "Breaking of Waves by an Opposing Current," by Yi-Yuan-Yu, <u>Transactions</u>, Amer. Geophysical Union, Vol. 33, No. 1, February, 1952.

A following current has the opposite effect: the waves become longer, lower, and less steep.

Applying these considerations to the effect of a curtain of air bubbles, leaving aside for the moment the fact that in that case the current is confined to a layer near the surface, it is seen that in front of the curtain the waves will meet the opposing current and therefore become shorter, higher and steeper. If the current is strong enough, nearly all the waves will break. With a somewhat slower current only the shorter waves of the spectrum will break and the longer waves will penetrate through the curtain. In that case there is a partial attenuation.

If the current is not strong enough, nearly all waves will pass through and thereafter, under the influence of the following current in the lee of the curtain,

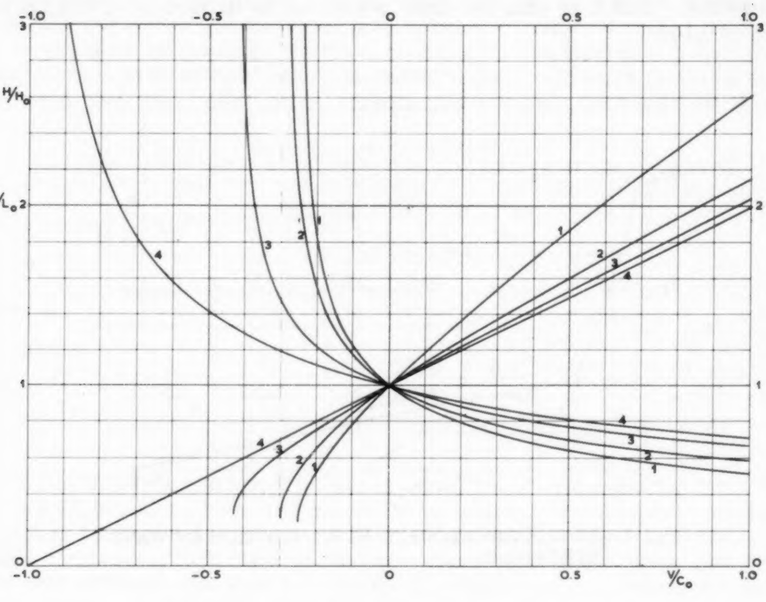

FIG. 3.—EFFECT OF CURRENT L/L_0 and H/H_0

become longer, lower and less steep. After leaving the area of the current they will resume approximately their initial characteristics. The process of being subject first to an increase in height and thereafter to a decrease in height, both with respect to the initial value, results in an apparent reduction, which easily may have been mistaken by some observers for a genuine reduction. Because of the difference in steepness the visual impression will be even stronger than the actual effect. In any case it adds considerably to the difficulty of obtaining accurate and reliable measurements, in particular if the experimental canal or the area of observation in the field is not long.

For waves that are long with respect to the water depth, the picture is rather different. If L:d > 25, the wave length virtually has no influence on the

wave velocity ($C = \sqrt{gd}$) and the result is that long waves can only be stopped by an opposing velocity $V = -C_0$. Eqs. 9 and 10 are

$$\frac{L}{L_0} = 1 + \frac{V}{C_0} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (9)$$

and

$$\frac{H}{H_0} = \left(1 + \frac{V}{C_0}\right)^{-\frac{1}{2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (10)$$

These relationships have also been presented in Fig. 3 (curves 4). In this range also the waves as a rule will break already at a lower velocity. However, a different criterium for breaking has to be applied, because these long waves never can obtain a steepness 1 in 7. They will break if their height increases to 0.78 d (d:H = 1.28), so that the limit steepness is defined by 0.78 d:L_0 instead of 0.143.

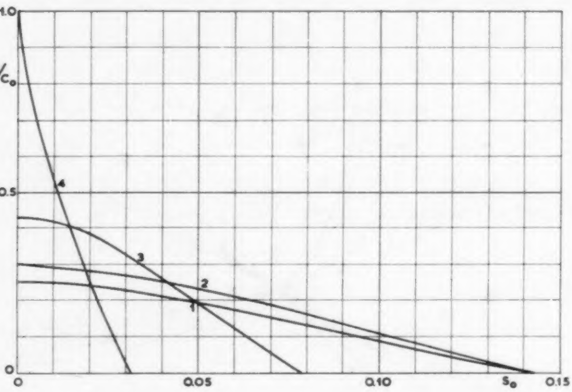

FIG. 4.—STOPPING VELOCITY AS A FUNCTION OF WAVE STEEPNESS

The stopping velocities for long waves listed in Table 8 and entered in Fig. 4 have been computed for L_0:d = 25. For these waves the initial steepness cannot exceed 0.78:25 = 0.0312 (1 in 32).

In actual practice the waves that have to be dealt with in protecting port entrances, construction, salvage and dredging operations and so forth, will most often be between the two extreme cases of L:d $\leqq$ 2 and L:d $\geqq$ 25. In this range the wave velocity is expressed by

$$C = \sqrt{\frac{g}{2\pi} L \tanh \frac{2\pi d}{L}} \quad \ldots\ldots\ldots\ldots\ldots \quad (11)$$

which makes it impossible to derive explicit expressions for the ratios H:H_0 and L:L_0. They can, however, be obtained by numerical computation, which

has been done for the cases L:d = 5 and L:d = 10. The results have been entered in Fig. 3 (curves 2 and 3).

As was to be expected, they are intermediate between the two extreme cases. In analogy to pure surface waves, the waves in the translation range show a limit velocity well below $-C_0$. For L:d = 5 it is $V = -0.3\ C_0$, for L:d = 10 it is $-0.43\ C_0$.

Also the stopping velocities for waves of different initial steepness have been determined again. For L_0:d = 5 the steepness 1 in 7 defines the limit for breaking, for L_0:d = 10 it is the depth. The maximum initial steepness in this case is 0.78:10 = 0.078 (1 in 12.8).

A few characteristic results are summarized in Table 8. Altogether the relative current velocities needed for stopping the L:d = 5 waves are not much higher than for the surface waves. This is not surprising, because one of the effects of the counter-current is to shorten the waves, as a result of which the L:d ratio decreases and the waves tend to become surface waves.

It would be misleading, however, to draw the conclusion that even fairly long waves can be stopped nearly as easily as short waves. It has to be borne in mind that the initial wave velocity C_0 is higher for the longer waves.

TABLE 8.—STOPPING VELOCITIES FOR DIFFERENT TYPES OF WAVES

Type of wave	L:d	Stopping value $V:C_0$ for waves with initial steepness						
		0	0.01	0.02	0.03	0.05	0.07	0.10
Surface	$\leqq 2$	0.25	0.25	0.24	0.22	0.185	0.15	0.085
Transition	5	0.3	0.29	0.28	0.26	0.23	0.185	0.105
	10	0.43	0.41	0.38	0.32	0.19	0.05	—
Long	$\geqq 25$	1.0	0.53	0.25	0.025	—	—	—

If as an example, a waterdepth of 10 m is taken, the wave lengths for L:d = 2, 5, 10, and 25, are 20 m, 50 m, 100 m and 250 m respectively, with wave velocities of 5, 6 m per sec, 8, 1 m per sec, 9, 3 m per sec and 9, 9 m per sec. Hence, the absolute stopping velocities in the four cases total to 1.4 m per sec, 2.4 m per sec, 4.0 m per sec and 9.9 m per sec and the stopping velocities for waves with an initial wave height of 2 m to 0.48 m per sec, 2.1 m per sec, 3.5 m per sec and 5.9 m per sec. Moreover currents covering the full depth are still being studied and, as shall be seen, the difference in effect between a surface current and a full depth current is greater for long waves than for surface waves.

Before going further into this difference a few preliminary conclusions may be drawn.

Waves can be stopped by an opposing current $V = \alpha\ C_0$, with α dependent on the L:d- ratio and the initial wave steepness.

The well known fact that the shorter and steeper the waves are, the easier they can be stopped, is clearly demonstrated. Waves with an initial steepness close to the breaking limit need only a low relative velocity, even if they are long. It has not appeared, that any particular wave length or L:d- ratio has a critical significance. Every wave can be stopped, provided the opposing current

is strong enough, that is, provided the volume of air injected is large enough. The fact that hitherto either in laboratory tests or field experiments no substantial reduction has been found for waves with L:d- ratio higher than 5 must be explained by the limited supply of air. Given certain conditions, there may be a practical limit to the length of waves that can be damped, but this limit is not significant from the physical point of view.

Waves passing through the curtain show an apparent reduction as a result of the increase in wave height in front of the curtain and the subsequent decrease in wave height behind. This may easily have distorted the result of observations, of which the conditions have not been clearly established. Also, in particular in the case of field observations the possibility that the curtain has acted as a filter, stopping short waves but allowing longer waves to pass through, adds to the uncertainity in interpretation of the observational data.

It now remains to discover if and how the effect of a surface current of the type generated by an air curtain can be related to the effect of a current over the entire depth. On this point we have some guidance from the theoretical calculations of Taylor.[22] He has treated both the action of a uniform current of a limited depth and of a current decreasing linearly from the surface to zero at a given depth. The second case is to a close approximation the distribution obtained with a bubble curtain (Fig. 2). Taylor had dealt only with waves in an infinite depth of water, so that his results are directly applicable only to surface waves (L:d = 2).

As long as this remains within this range, Taylor's results indicate that with a uniform distribution the stopping velocity is only little more than $\frac{1}{4}$ C_0, as long as the thickness of the current is more than about 1/15 of the wave length. That means that such a surface current has the same effect as an equal current over the full depth.

In the case of triangular flow distribution for waves with L = 2 d the required surface velocity is 1.25 to 1.3 times more, for example, 0.3 to 0.4 C_0. The ratio is smaller for still shorter waves and for short waves it approaches unity.

These results correspond with the fact that most of the energy of surface waves is concentrated near the surface, so that the portion of the countercurrent nearest the surface has the highest effect.

For waves in the transition region (L:d = 2 to 25) it may be expected that the ratio $V_m: V$ (V_m = surface velocity of triangular current, V = full depth uniform velocity) will increase with increasing L:d- ratio to a miximum which is also valid for long waves.

In order to arrive at an estimate for this maximum, it might be supposed that the same total flow of momentum is required. In that case, and with a thickness of the current of 0.25 d, it would be found that, $V_m = 2\sqrt{3}$ V and, because for long waves the uniform stopping velocity equals the wave velocity $V_m = 2\sqrt{3}\, C_0$ or about 3.5 C_0.

It may be assumed, however, that a much lower surface velocity, for example from $V_m = \alpha C_0$ upward, without causing the waves to break completely, will already have an attenuating effect. A counter velocity $V_m > \alpha C_0$ presumably will bring about a partial breaking, while allowing part of the wave energy to pass through underneath. Even so the flow and therefore the volume of air required for a substantial reduction will be excessively large.

In this stage it is possible to arrive at an estimate of the volume of air and the consumption of power required in different conditions. For that purpose

again consider the waterdepth of 10 m and waves of 20 m and 50 m length with wave height 2 m. The stopping velocities in case of a full depth current have been conducted earlier to be respectively 0.48 m per sec and 2.7 m per sec. According to Taylor's analysis the corresponding surface velocities V_m may be estimated at approximately 1.35 x 0.48 = 0.65 m per sec and 2.1 x 2.1 = 4.4 m per sec. Applying Bulson's formula (1) one finds for the volume of air per m length of screen 0.009 m^3 per sec and 2.8 m^3 per sec at 10 m depth, or under atmospheric pressure 0.018 m^3 per sec and 5.6 m^3 per sec. The corresponding power per m length of screen needed for compressing the air to 2 atmospheres (10 m depth), not counting friction in pipes and other losses, would be about 1.7 hp for the 20 m wave, but already 520 hp for the 50 m wave.

It is, however, rather doubtful if for the high surface velocity required for the 50 m wave the relationships established by Bulson can still be applied. The result for the 20 m wave is in the order of magnitude used in some of the full scale tests. It may be observed in passing that the power continued in the counter-current with V_m = 0.65 m per sec amounts to about 0.1 hp, so that the efficiency of the airlift as a current generator is only 6%.

In the writer's opinion an attempt to distill a more coherent picture from the results of the past investigations should be based on an analysis of the type given in the foregoing. It will then be necessary in the first place to subject the information presented by the different authors to a critical examination, possibly followed by the elimination or re-evaluation of those observations that obviously or probably reflect an apparent rather than a genuine reduction in wave height. Also in the case of field observations the reductions measured should be related to the different components of the wave-spectrum rather than to the significant wave because of the selective action of an air curtain with regard to waves of different length.

For every observation the volume of air should be translated into surface velocity V_m of the horizontal current by means of Bulson's relationship Eq. 3 or Eq. 5. Then the reduction coefficients obtained after the screening process outlined previously should be classified according to the ratios $V_m:C_0$, L:d and H:L, and it should be tried to establish relationships between the reduction-coefficient and these parameters. Only on such a basis can a sound and rational evaluation of the feasibility of the method be obtained and reliable design criteria for given conditions be established.

It is of course somewhat doubtful if the available information, mostly originating from researches set up without a view to this type of procedure, can be expected to yield a rich harvest. Quite probably it will appear necessary, in order to arrive at really pertinent results to undertake fresh investigations, designed to produce observational data suitable for processing in this manner.

In such an investigation also the question of the most effective manner of producing the air curtain (size of bubbles, possible scale-effect) might be included.

It is to be expected on the strength of the accumulated knowledge of the subject, vague as it is, that the occasions where a curtain of air bubbles can be used economically for protection against waves will not be numerous. On the other hand use of such a curtain for other purposes, such as increased mixing to combat the intrusion of salinity and suspended silt, seems to have distinct possibilities. Therefore it may well happen that installations for the generation of an air curtain, either in a fixed position or transportable, will

be constructed in the near future. Such installations might then be available also for wave protection, should the occasion arise and the conditions be favorable. In this light a better understanding of the process, even if the outlook is not too hopeful, may be valuable.

GROINS ON THE SHORES OF THE GREAT LAKES[a]

Discussion by A. C. Rayner and R. L. Rector

A. C. RAYNER[15] and R. L. RECTOR.[16]—The author has presented much data of interest concerning groins on the Great Lakes shores. As indicated in the paper this type of shore protection is, in general, successful only where the alongshore movement of littoral drift is reasonably plentiful. The natural condition along much of the Great Lakes shore line is just the opposite; beach building material is generally scarce and alongshore transport is low. The need of plentiful littoral drift is even more critical for a beneficial performance of permeable groins. It is, thus, not surprising that C. E. Lee has found in his sample areas that only 13% of the groins built were of the permeable type. It is also noted that more than half of the groins described are 100 ft or less in length. Groins of such limited length would normally not be expected to be effective in building an adequate protective beach even under more favorable conditions. The principal benefit of such short structures is to retain a narrow protective beach from the beach-size material eroded from the bluffs, and, thus, offer partial protection or retard to some degree further recession of those bluffs.

In considering the data on groin costs presented by Lee, it should be noted that the figures for cost per linear foot of groin are for the most part derived for structures under 200 ft in length. The most costly part of a groin is the offshore section. In general fully effective groins are more than 200 ft in length, the extra length of course being in deeper water and thus substantially higher in cost.

Regarding the use and effectiveness of groins in the Great Lakes area, it may be pointed out that in very few instances has the Beach Erosion Board recommended construction of groins without artificial placement of beach fill, and in no case has that Board found it appropriate to recommend construction of permeable groins. The artificial filling of the groin system tends to prevent the adverse effect on the downdrift shore inherent in groin operation, that is, the reduction of supply to and consequent erosion of the downdrift shore as a result of impoundment of the littoral drift by the groin system. The real justification for groins is considered to be their effectiveness in reducing alongshore losses of artificially placed protective beaches, especially where the unit cost of sand for periodic beach nourishment is high.

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[a] May 1961, by Charles E. Lee (Proc. Paper 2819).

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PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU); and Waterways and Harbors (WW), divisions. Papers sponsored by the Department of Conditions of Practice are identified by the symbols (PP). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper number are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 2703 is identified as 2703(ST1) which indicates that the paper is contained in the first issue of the Journal of the Structural Division during 1961.

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NOVEMBER: 2637(ST11), 2638(ST11), 2639(CO3), 2640(ST11), 2641(SA6), 2642(WW4), 2643(ST11), 2644(HY9), 2645(ST11), 2646(HY9), 2647(WW4), 2648(WW4), 2649(WW4), 2650(ST11), 2651(CO3), 2652(HY9), 2653(HY9), 2654(ST11), 2655(HY9), 2656(HY9), 2657(SA6), 2658(WW4), 2659(WW4)[c], 2660(SA6), 2661(CO3), 2662(CO3), 2663(SA6), 2664(CO3)[c], 2665(HY9)[c], 2666(SA6)[c], 2667(ST11)[c].

DECEMBER: 2668(ST12), 2669(IR4), 2670(SM6), 2671(IR4), 2672(IR4), 2673(IR4), 2674(ST12), 2675(EM6), 2676(IR4), 2677(HW4), 2678(ST12), 2679(EM6), 2680(ST12), 2681(SM6), 2682(IR4), 2683(SM6), 2684(SM6), 2685(IR4), 2686(EM6), 2687(EM6), 2688(EM6), 2689(EM6), 2690(EM6), 2691(EM6)[c], 2692(ST12), 2693(ST12), 2694(HW4)[c], 2695(IR4)[c], 2696(SM6)[c], 2697(ST12)[c].

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JANUARY: 2698(PP1), 2699(PP1), 2700(HY1), 2701(SA1), 2702(SU1), 2703(ST1), 2704(ST1), 2705(SU1), 2706(HY1), 2707(HY1), 2708(HY1), 2709(PO1), 2710(HY1), 2711(HY1), 2712(ST1), 2713(HY1), 2714(PO1), 2715(ST1), 2716(HY1), 2717(SA1), 2718(SA1), 2719(SU1)[c], 2720(SA1)[c], 2721(ST1), 2722(PP1)[c], 2723(PO1)[c], 2724(HY1)[c], 2725(ST1)[c].

FEBRUARY: 2726(WW1), 2727(EM1), 2728(EM1), 2729(WW1), 2730(WW1), 2731(EM1), 2732(SM1), 2733(WW1), 2734(SM1), 2735(EM1), 2736(EM1), 2737(PL1), 2738(PL1), 2739(PL1), 2740(PL1), 2741(EM1), 2742(ST2), 2743(EM1), 2744(WW1), 2745(WW1), 2746(SM1), 2747(WW1), 2748(EM1), 2749(WW1), 2750(WW1)[c], 2751(EM1)[c], 2752(SM1)[c], 2753(PL1)[c], 2754(ST2)[c], 2755(PL1).

MARCH: 2756(HY2), 2757(IR1), 2758(AT1), 2759(CO1), 2760(HY2), 2761(IR1), 2762(IR1), 2763(HY2), 2764(HY3), 2765(HY2), 2766(HW1), 2767(SA2), 2768(CO1), 2769(IR1), 2770(HY2), 2771(SA2), 2772(HY2), 2773(CO1), 2774(AT1), 2775(IR1), 2776(HY3), 2777(HY2), 2778(SA2), 2779(ST3), 2780(HY2), 2781(HY2)[c], 2782(HW1)[c], 2783(SA2)[c], 2784(CO1), 2785(CO1)[c], 2786(IR1)[c], 2787(ST3)[c], 2788(AT1)[c], 2789(HW1).

APRIL: 2790(EM2), 2791(SM2), 2792(SM2), 2793(SM2), 2794(SM2), 2795(SM2), 2796(SM2), 2797(SM2), 2798(EM2), 2799(EM2), 2800(EM2), 2801(EM2), 2802(ST4), 2803(EM2)[c], 2804(SM2)[c], 2805(ST4)[c].

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JULY: 2847(PO2), 2848(SU2), 2849(HY4), 2850(PO2), 2851(HY4), 2852(PO2), 2853(SU2), 2854(HY4), 2855(PO2), 2856(PO2), 2857(PO2), 2858(SA4), 2859(SU2), 2860(SA4), 2861(PO2), 2862(SA4), 2863(HY4), 2864(HY4), 2865(HY4), 2866(HY4), 2867(HY4), 2868(PO2)[c], 2869(SA4)[c], 2870(SU2)[c], 2871(HY4), 2872(HY4)[c], 2873(SU2), 2874(SA4).

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OCTOBER: 2950(PP2), 2951(PP2), 2952(PP2), 2953(ST7), 2954(SM5), 2955(ST7), 2956(ST7), 2957(ST7), 2958(ST7), 2959(SM5), 2960(EM5), 2961(EM5), 2962(ST7), 2963(ST7), 2964(EM5), 2965(SM5), 2966(SM5), 2967(ST7), 2968(ST7), 2969(ST7), 2970(ST7), 2971(SM5), 2972(SM5)[c], 2973(EM5)[c], 2974(ST7)[c], 2975(PP2).

NOVEMBER: 2976(HY6), 2977(HY6), 2978(HY6), 2979(PO3), 2980(WW4), 2981(HY6), 2982(HY6), 2983(HY6), 2984(HY6), 2985(SA6), 2986(SA6), 2987(WW4), 2988(WW4), 2989(WW4), 2990(HY6), 2991(WW4), 2992(HY6), 2993(PO3), 2994(WW4), 2995(PO3), 2996(HY6), 2997(SA6), 2998(PO3), 2999(PO3)[c], 3000(HY6)[c], 3001(SA6), 3002(SA6), 3003(SA6)[c], 3004(WW4)[c].

c. Discussion of several papers, grouped by divisions.

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